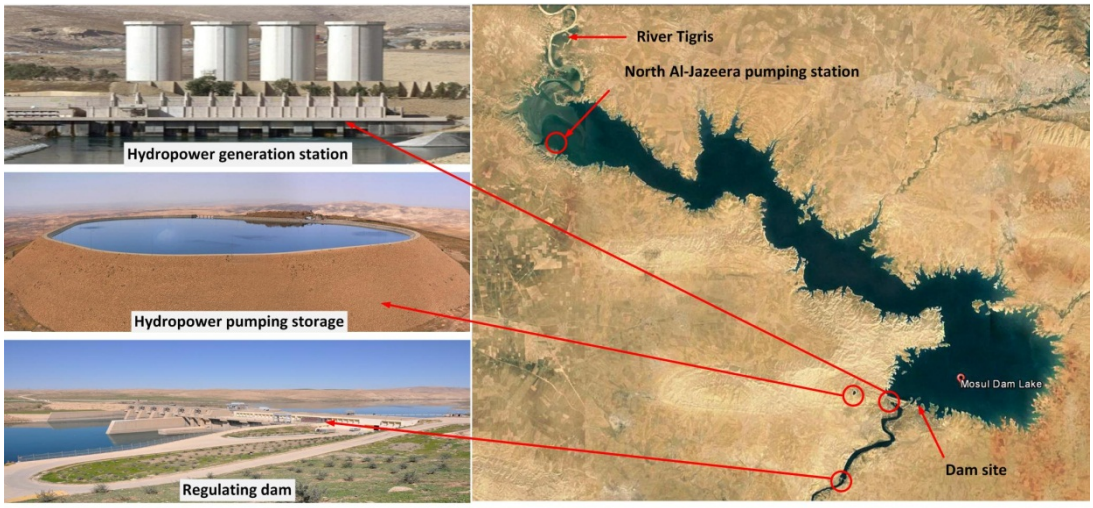
LULEÅ UNIVERSITY OF TECHNOLOGY

ENGINEERING AND GEOLOGICAL INVESTIGATION OF THE MOST DANGEROUS DAM IN THE WORLD



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PREFACE

Mosul Dam is located on the Tigris River in north western part of Iraq; approximately 60 km northwest of Mosul city and 80 Km from Syrian and Turkish borders. It is a multipurpose project for irrigation, flood control and hydropower generation. It is 113 m in height, 3.4 km in length, 10 m wide in its crest and has a storage capacity of 11.11 billion cubic meters. The water surface area of the reservoir at the beginning of the dam operation was 380 km^2 with a storage capacity of 11.11 km^3 at the maximum operation level 330 m (a.s.l.) including 8.16 km^3 live storage and 2.95 km^3 dead storage. It is an earth fill dam, constructed on bedrocks of the Fatha Formation, which consists of gypsum beds alternated with marl and limestone, in cyclic nature. The thickness of the gypsum beds attains 18 m; they are intensely karstified even in foundation rocks. This has created number of problems during construction, impounding and operation of the dam.

Construction work in Mosul Dam started on January 25th, 1981. Swiss Consultants Consortium was asked to be the consultants for Mosul Dam project. A consortium of German and Italian companies (GIMOD) was asked to execute the civil and steel work of the project in 1980. While electromechanical plant of the power station was given to the Japanese company (Toshiba) on the condition that the capacity of the plant will be 750 MW. The electromechanical plant contract for the regulation dam was awarded to (Elin Union) from Austria, while the electromechanical plant for the pump storage scheme was given to (G.I.E) from Italy. The total cost of the development was estimated at 2.6 billion US$ at the prices level of 1985and the dam started operating on 24th July, 1986. After impounding in 1986, new seepage locations were recognized. Grouting operations continued and various studies were conducted to find suitable grout or technique to overcome this problem. The seepage due to the dissolution of gypsum and anhydrite beds raised a big concern about the safety of the dam and its possible failure. This problem was kept in a small closed circle within the Iraqi Ministry of Water Resources (previously Ministry of Irrigation) till the US Army Corps of Engineers conducted a study on Mosul Dam for the period June, 2004 to July, 2006 and highlighted the possibility of the dam failure. News media had highlighted this concern in 2014 when ISIS occupied the dam site area. It was reported that about 500,000 will lose their lives in case of Mosul Dam failure. Loses of property and destruction of the infrastructure of the main cities downstream the dam will be enormous and it will reach the capital city Baghdad.

In this study, the problem of the stability of Mosul Dam will be thoroughly discussed through the available reports and the experiences of the authors. Finally, recommendations are given to avoid the catastrophe of Mosul Dam failure.

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CHAPTER 1: GENERAL GEOLOGY

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1. Introduction

Mosul Dam is one of the most important strategic projects in Iraq for management of its water resources. It is considered the second biggest dam in the Middle East. The project was constructed on the Tigris River in the north of Iraq, located 60 km north west of Mosul city and 80 km from the Syrian and Turkish borders at 4056066 m N northing and 305356.69 m E easting (Iraqi Ministry of Water Resources, 2012) (Fig. 1).

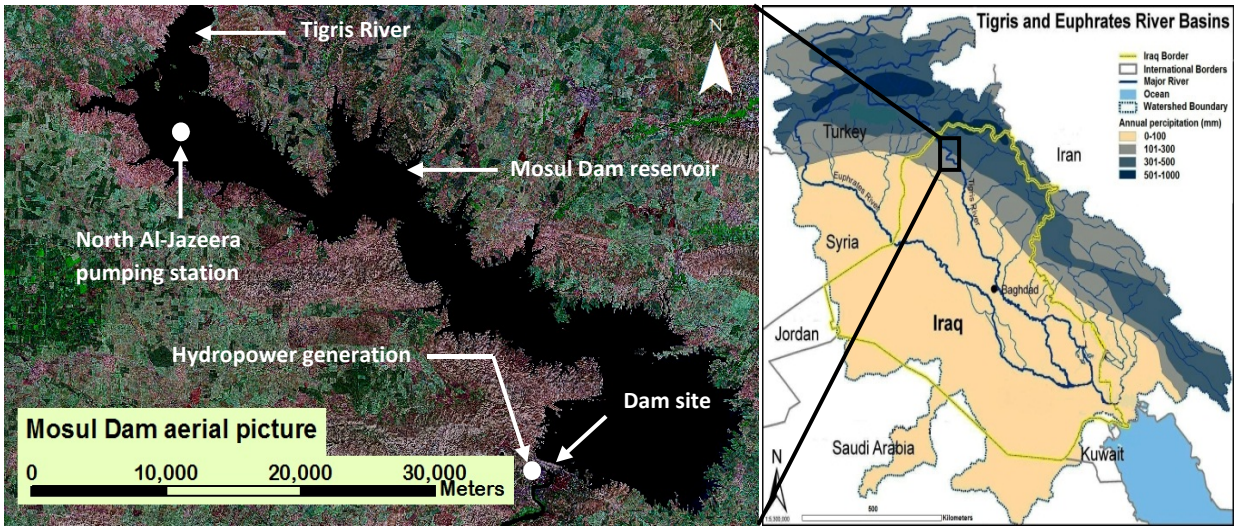

Figure 1: Location of Mosul Dam with main facilities.

Construction of Mosul Dam began on January 25th, 1981. The dam is a multipurpose project and started operating on July 7th, 1986 to provide water for three irrigation projects, flood control and hydropower generation. The dam is 113 m high and 3650 m long including the spillway. The top width is 10 m at 341m (a.s.l.) crest level. The dam is earth fill type with a mud core. The upstream side is faced with rock (Iraqi Ministry of Water Resources, 2012). The maximum, normal and dead storage levels of its reservoir are 335, 330 and 300 m (a.s.l.) respectively. The dam was designed to impound 11.11 km^3 of water at normal operation level, including 8.16 and 2.95 km^3 of live and dead storage respectively (Fig. 2).

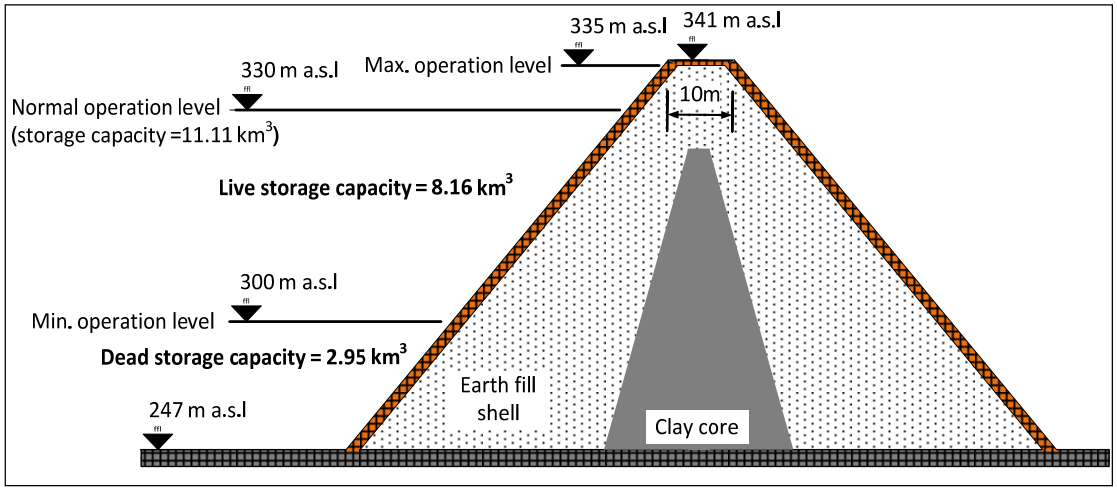

Figure 2: Schematic diagram of Mosul Dam cross section.

The dam has a concrete spillway located on the left abutment. The crest elevation of the spillway is 330 m (a.s.l.) and its length is 680 m. The spillway has five radial gates measuring 13.5 m×13.5 m giving a discharge of 12600 m^3 sec^{-1} at the maximum reservoir level of 338 m (a.s.l.) (Iraqi Ministry of Water Resources, 2012). In this research, the general geology of the dam project area including the reservoir is discussed

2. Geomorphology

The topography of the Mosul Dam and reservoir area is characterized by hilly terrain that rises to low mountainous area; representing physiographically Low Mountainous Province (Sissakian and Fouad, 2012). The mountains form anticlines, which trend mainly in NW-SE direction and changes westwards almost to E-W direction.

The main geomorphological units in the concerned area are of Structural-Denudational origin; the main units are the anticlinal ridges, which form the limbs of the existing anticlines, either formed by limestone or gypsum (Fig.3). Another geomorphological form is the flat irons formed either in limestone beds or gypsum (Fig.3); however in the outer areas of the limbs, sandstone beds form cuestas and Hogbags; due to their alternation with soft claystone beds (Sissakian, 1995).

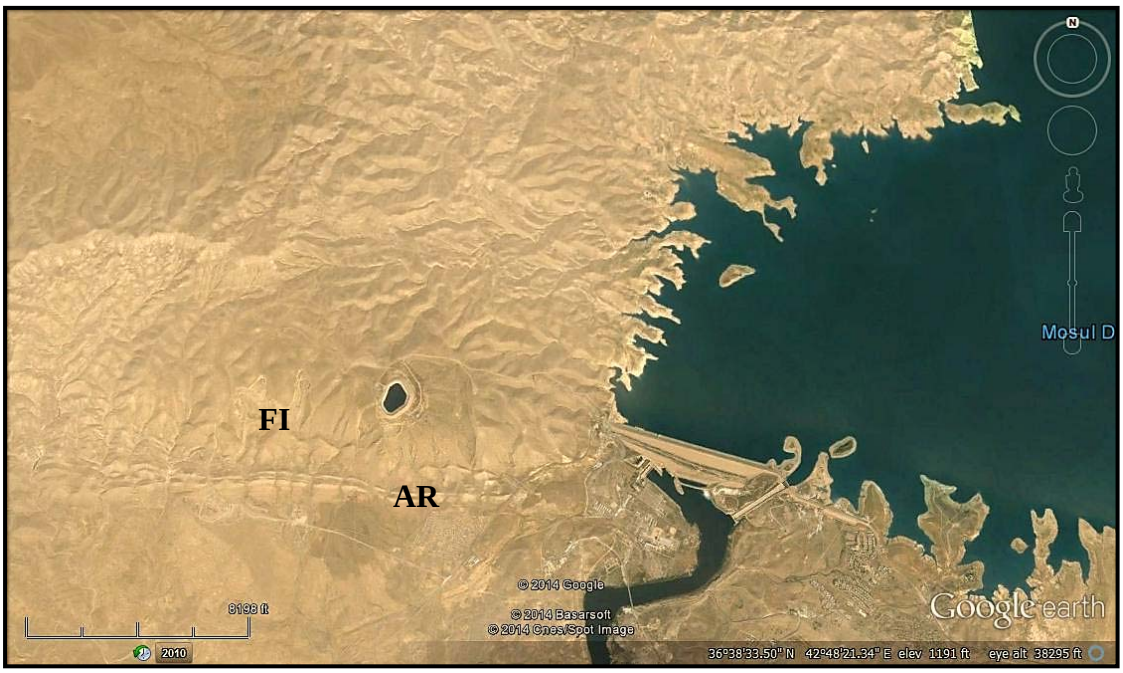

Figure 3: Google Earth image of the Mosul Dam site, located within Butma East anticline. **AR**= Anticlinal ridge, **FI**= Flat iron.

Another significant geomorphological aspect is the karstification, which is densely developed in the dam site and reservoir area (Fig.4) (Sissakian, 1995; Jassim et al., 1997; Sissakian and Al-Mousawi, 2007; Sissakian et al., 2014). The main karst feature is the sinkholes, which are developed in limestones and gypsum beds. Those developed in limestone beds have regular forms, either with circular or elliptical apertures; the diameter ranges from (< 1-20) m, whereas the depth rages from (< 1-15) m, whereas those developed in gypsum beds have irregular

apertures with clear dissolving indications, the diameter ranges from (< 1-3) m, whereas the depth rages from (< 1-8) m (Sissakian, 1995; Sissakian and Al-Mousawi, 2007).

Figure 4: Enlarged Google Earth image showing many sinkholes (Dark colored circles).

3. Tectonics and Structural Geology

Mosul Dam site and reservoir area are located within the Cham – Chamal Subzone of the Low Folded Zone; within the Unstable Shelf of the Arabian Plate (Al-Kadhimi et al., 1996; Jassim and Goff, 2006). Fouad (2012a); however, using the plate tectonics theory updated the tectonic frame work of Iraq, and consequently considered that the dam site and reservoir area are located within the Low Folded Zone, within the Outer Platform of the Arabian Plate.

The Low Folded Zone of Iraq is characterized by long anticlines and narrow synclines, which are usually filled with Quaternary sediments, whereas the cores of the anticlines are occupied by Miocene rocks, mainly Middle Miocene. Mosul Dam is located within Butma East anticline, which has almost E – W trend with steeper southern limb (Fig.3).

The reservoir area of Mosul Dam includes many anticlines, which surround the limits of the reservoir; however, many others are located nearby to the reservoir and the dam site (Fig.5). Some of them exhibit strange shapes and local dislocations due to a major deep seated fault called Sasan – Bekhair Fault (Fouad, 2012b), which passes NW of the dam with clear dislocation of the axes and beds of many anticlines (Fig.6). Moreover, many other small faults of different types and different dislocations were mapped in the dam site and reservoir area (Taufiq and Domas, 1977). It is worth mentioning that all structural disturbances have no any significant effect on the dam.

Buday and Jassim (1987) and Al-Kadimi et al. (1996) reported about deep seated fault called "Sinjar – Dohuk – Amadiya Fault" to exist in the reservoir area; however, Fouad (2012a) denied the presence of this fault. No surface indication is present to confirm the presence of the claimed fault.

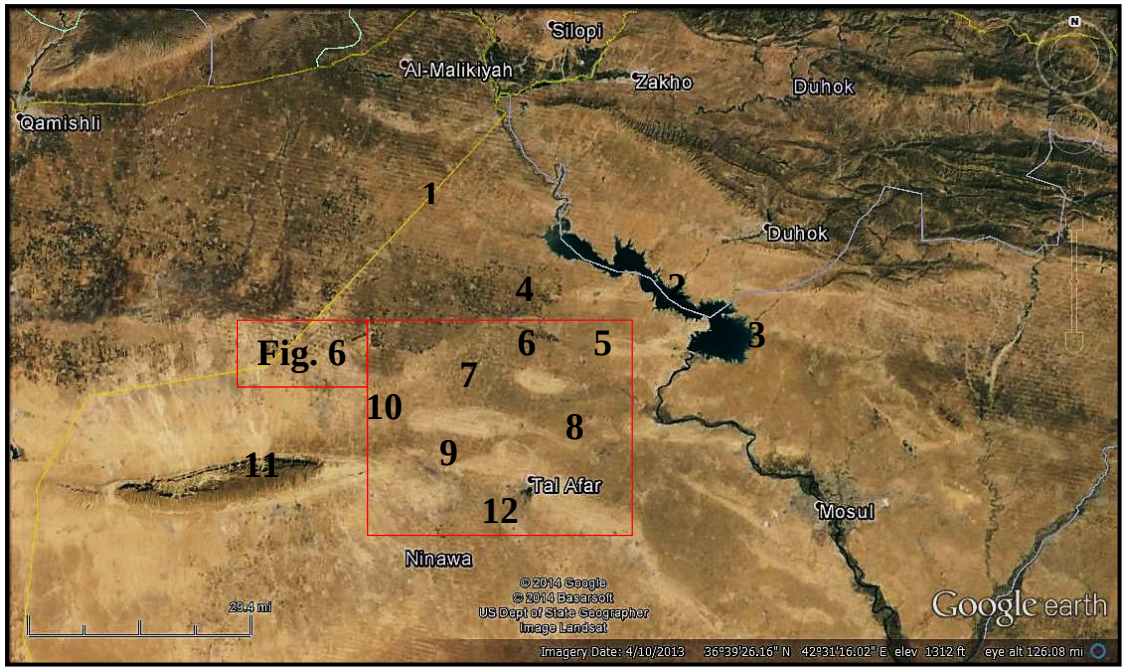

Figure 5: Google Earth image of Mosul Dam site and reservoir area showing the existing anticlines: 1: Mashoura, 2: Dahqan, 3: Qand, 4: Ain Zala, 5: Butma East**,** 6: Butma West**,** 7: Qusair, 8: Alan, 9: Sasan, 10: Ishkaft, 11: Sinjar, and 12: Sheikh Ibrahim

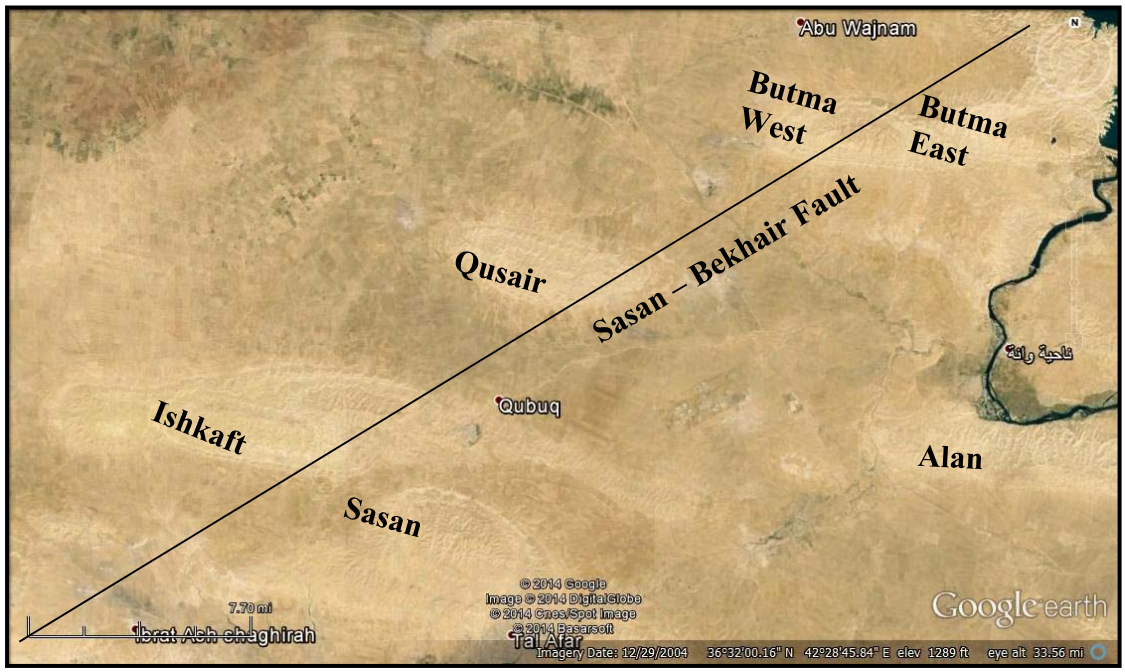

Figure 6: Google Earth image showing dislocated anticlines; along Sasan – Bekhair Fault

4. Stratigraphy

The oldest exposed rocks in the dam site belong to the Fatha Formation (ex-Lower Fars) of Middle Miocene age; however, in the reservoir area, the oldest exposed rocks belong to the Pila Spi Formation of Late Eocene age (Taufiq and Domas, 1977; Sissakian, 1995; Sissakian and Fouad, 2012) (Fig.7). The exposed formations in the dam site and reservoir area are described briefly hereinafter.

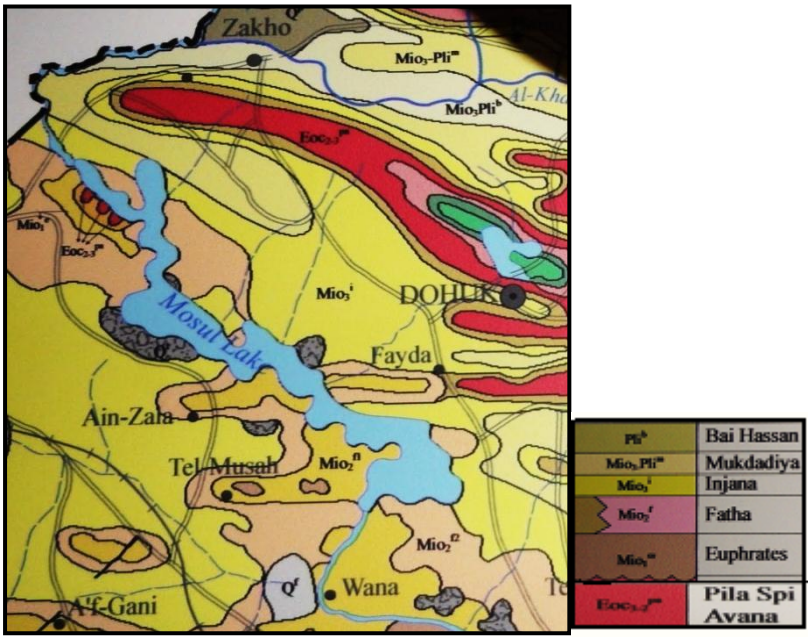

Figure 7: Geological map of Mosul Dam site and reservoir area (After Sissakian and Fouad, 2012).

4.1 Pila Spi Formation (Late Eocene)

Small outcrops of the Pila Spi Formation exist in the extreme northwestern side of the reservoir (Fig.7). The formation consists of well bedded, dolomite, dolomitic limestone, limestone and rare marl (Taufiq and Domas, 1977; Sissakian and Saeed, 2012). The exposed thickness of the formation is few meters only.

4.2 Euphrates Formation (Early Miocene)

The Euphrates Formation is exposed in the core of some anticlines in the reservoir area (Fig.7), as well in the foundations of the dam. The formation consists of well bedded, hard and fossiliferous limestone, marly limestone with less abundant dolomitic limestone (Taufiq and Domas, 1977; Sissakian and Saeed, 2012). Some of the limestone beds exhibit karstification as indicated by the presence of sinkholes. The thickness of the formation in nearby areas ranges from (15 – 50) m (Taufiq and Domas, 1977; Mohi Ad-Deen et al., 1977).

4.3. Jeribe Formation (Middle Miocene)

The Jeribe Formation is not recorded to be exposed in the dam site and reservoir area (Taufiq and Domas, 1977; Sissakian and Fouad, 2012). However, the formation is recorded in the foundations of the dam (Sissakian et al., 2014), and to be exposed in oil well Butmah 2 (I.P.C.,

1964). The formation consists of well bedded, hard and fossiliferous limestone; highly resembling to the Euphrates Formation (Sissakian and Saeed, 2012). No thickness record exists in the dam site; however, in some drilled oil wells in near surroundings is about 60 m, Oil wells Ain Zala 22, Mushorah 1 and 33 m in Butmah 2 (I.P.C., 1964).

4.4. Fatha Formation (Middle Miocene)

The Fatha Formation is widely exposed in the dam site and reservoir area (Fig.7). The formation is characterized by cyclic deposits, the thickness of the formation is variable; in Butmah is 392 m, in Ain Zala 325, in Mashourah 320 m, in Ishkaft 350 m (Sissakian and Al-Jibouri, 2012). The formation consists of two members (Taufiq and Domas, 1977; Sissakian, 1995; Sissakian and Fouad, 2012), these are:

4.4.1. Lower Member

The Lower Member of the Fatha Formation consists of cyclic deposits, each cycle consists of green marl, limestone and gypsum; however, in some cycles one of the three constituents may be absent. The abutments of the dam are located within this member. The rocks of this member are highly karstified, not only in the dam site and the reservoir area, but else-where in Iraq.

4.4.2. Upper Member

The Upper Member of the Fatha Formation consists of cyclic deposits, each cycle consists of green marl, red claystone limestone and gypsum; in the uppermost part reddish brown sandstone may be present too; however, in some cycles one of the three constituents may be absent. This member covers majority of the reservoir area.

4.5. Injana Formation (ex-Upper Fars) (Late Miocene)

The Injana Formation is exposed in the eastern, northeastern and some parts of the northern banks of the reservoir (Fig.7). The formation consists of fining upwards cyclic deposits of reddish brown sandstone, siltstone and claystone. The thickness of the formation is variable ranges from (200 – 330) m (Sissakian and Al-Jibouri, 2012).

4.6. Mukdadiya Formation (ex-Lower Bakhtiari) (Late Miocene – Pliocene)

The Mukdadiya Formation is exposed in the eastern bank of the reservoir (Fig.7). The formation consists of fining upwards cyclic deposits of grey sandstone, siltstone and claystone; some of the sandstone beds are pebbly. The thickness of the formation is variable ranges from (100 – 230) m (Sissakian and Al-Jibouri, 2012).

5. Seismicity

Although Mosul Dam and near surroundings are located far from Zagros Active Seismic Zone, but still many earthquakes were recorded in areas surrounding the project area. According to Al-Sanawi and Al-Qasrani (2003) and Al-Sanawi (2006), the dam area and the reservoir are located in Zones IX and VIII of Historical Isointensity Map, respectively and in Zone V of Seismic Isointensity Map, which means zone of minor damages.

The seismicity index (ST) is related to the frequency of occurrences of earthquakes magnitudes 4.0. The dam site area is located in region of ST = 0.1 of Seismic Index Map, which means the area, suffers an earthquake with magnitude ≥ 4.0 every five to ten years in an area with diameter of 100 Km. Moreover, it is located in Zone (0.4-0.5) of Seismic Acceleration Map with design period of 100 years (Fig.8).

According to Al-Kadhimi et al. (1997) and Al-Sanawi (2006), there are many epicenters of (3.6-5.3) degrees on Richter scale nearby the dam site. Moreover, during the year 2014, many earthquakes were recorded northwest, north and northeast of Mosul city, but all of them are within magnitudes of less than 4 degrees on Richter scale (Mohammed et al., 2014).

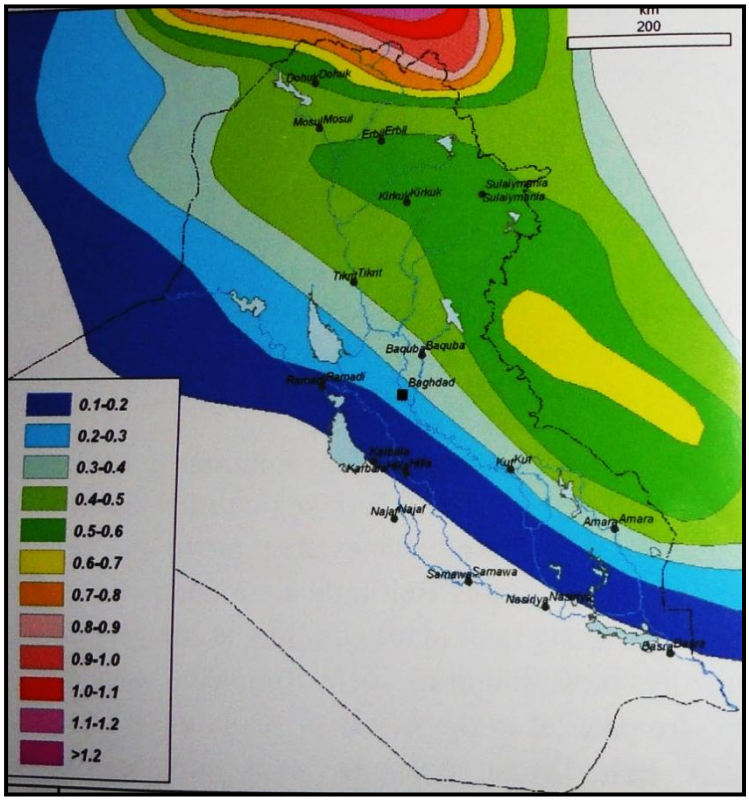

Figure 8: Seismic acceleration map with design period of 100 years (After Al-Sanawai, 2006).

In addition to the above, it is noteworthy to mention that the Swiss Consultants carried out Seismicity study for Mosul Dam (Swiss Consultants, 1982). This report was reviewed by Washington Group International in 2005. The main remarks about the Swiss Consultant (1982) report were:

I. The SC report seemed to be well prepared and contained the information available up to 1985. But new data were generated from 1985 to 2005 with respect to plate tectonics.

II. Very little information on local and regional geology was given in SC report. (The answer to this remark is that SC had already compiled this information in the planning report and there was no need to repeat again).

III. Much of the information given was “attenuation relationships” which were developed for the earthquake data and seismic sources areas that were researched for the project. The source zones were developed for ground motion was based on plots of earthquakes epicenters as:

1. Low seismicity zone (40 Km radius around site).
2. Moderate seismicity zone (from 40 km to 150km radius around site).
3. High seismicity zone of Zagros Mountains (from 150km to 250km radius around site).

The zones were separated so that different attenuation relationships could be developed for the different data and distances involved.

On the basis of this information, a design earthquake with 280 to 450 years return period was developed with recommended ground peak acceleration (PGA) of 0.15g. The maximum credible earthquake (MCE) with a return period on the order of 10000 years was recommended to be 0.25 g.

The remarks stated by Washington Group International in 2005 were based on the data they reviewed from the internet. These data were addressing the following topics:

A. Regional Tectonics: Iraq is located on the Arabian subcontinent tectonic plate which is subduction zone under Anatolian and Iranian plates along the Zagros-Taurus zone at the NW boundary of the plate. So, north and north eastern parts of Iraq have the highest seismicity. Fig.9 shows most of the seismicity recorded in the Middle East.

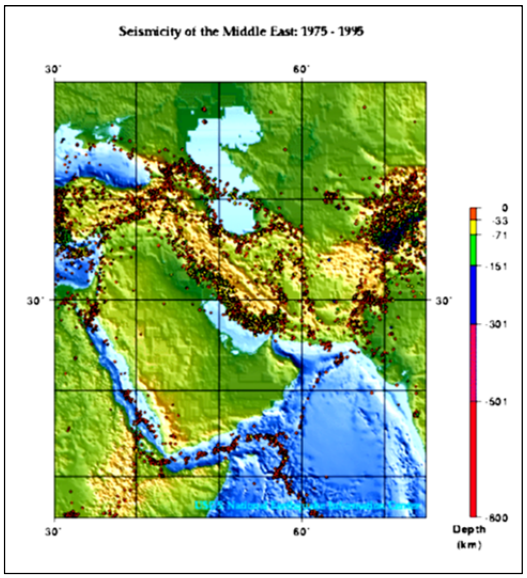

Figure 9: Seismicity of the Middle East (Washington Group International, 2005).

B. Geological Setting: The tectonics of Iraq has been divided as follows:

1. The stable zone.
2. The Mesopotamia zone (between the Tigris and Euphrates rivers).
3. The Simply folded zone (within which the dam is located); characterized by folds and thrust faults.

4. Imbricate zone (series of thrust zones).
5. The Thrust zone (complex thrust structures as the Main recent fault of Iran (on the Iraq-Iran border on which major earthquakes have occurred).

C. Seismicity: Historic seismicity shows 91% of earthquake events in Iraq are between 4 and 5.5 occurring at immediate depths from 30 to 90 km. Reviewing the earthquake records of SC report by a new review with the records of the US Geological Survey, National Earthquake Information Centre (2005), shows the highest magnitude recorded within 200 km of the site is Magnitude 6+ at a distance of about 121 km.

The only recent probabilistic seismic hazard map produced was by the Global Seismic Hazard Assessment Program (GSHAP), but it was possible to accesses only a portion of this map through USGS web site due to restriction by password (Fig.10).

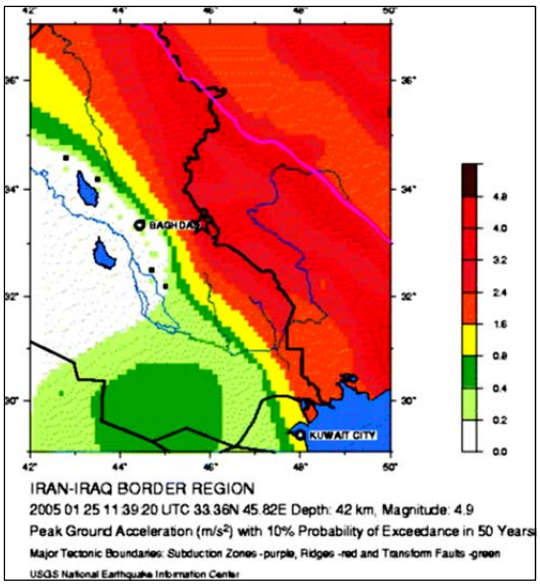

Figure 10: Probabilistic Seismic Hazard Map for a Portion of Iraq/Iran (US Geological Survey, National Earthquake Information Center, 2005).

Notes: 1. Site is located in the far Northwest portion of the Map.

2. Map corresponds to the 475-year return period.

3. Peak ground acceleration at the site corresponds to 1.6 to 2.4 m/s2. This value corresponds to 0.16 to 0.24g.

4. Map taken from National Earthquake Information Center

This map (Fig. 10) shows that probabilistic seismic hazard with 10% probability of being exceeded in 50 years, or about 475 years return period. The purple-pink line in the map shows location of the subduction zone. From the map the site (at North West corner) will be subjected to peak ground acceleration of 0.16g to 0.24g for this return period which is similar to the MCE recommendation In the SC report. The current practice in dam design is for MCE events that are closer to 2500-10000 years return period, depending on the significant hazard.

Finally, Washington Group International (2005) concluded and recommended the following:

1. The SC report was well prepared but it did not reflect current plate theory with respect to earthquakes and seismicity.

2. The site is located near the boundary of the subduction zone (Arabian plate colliding against Iran plate).It is in the simply folded zone, an area of folded rock and thrust faults that accommodate some of the compression.
3. Based on this the dam can be subjected to ground accelerations from two sources, one is the large earthquakes that occur in the subduction zone. The second source is related to background earthquakes close to site (which may result from surface faults).
4. This assessment report is based on data available on the internet (on Iraq). Additional information may be obtained from more complete review of data.
5. SC report recommended PGA of 0.15g for design earthquake based on 280 to 450 years return period and recommended PGA of 0.25g with the MCE with return period of 10000year. Available current information on the internet from GSHAP shows recommended PGA for the 475 year return period 0.16 g to 0.25 g.
6. Recommended additional work to this project to identify all potential sources of earthquake ground motion. In view of this a probabilistic seismic hazard study of the area should be conducted for the designated return period which reflects the hazard assessment for the dam.

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CHAPTER 2: THE PROJECT

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1. Introduction

The rivers Tigris and Euphrates form the main water resources of Iraq. Most of the water from these rivers comes from Turkey (71%) followed by Iran (6.9%) and Syria (4%). The remainder, only 8%, is from internal sources (Iraqi Ministry of Water Resources, 2014; Al-Ansari et al., 2014 and 2015). The average annual flow of the Rivers Euphrates and Tigris is estimated to be about 30 km^3 (which might fluctuate from 10 to 40 km^3) for the former and 21.2 km^3 for the latter when they enter Iraq. Tigris River tributaries in Iraq contribute 24.78 km^3 of water and there is about 7 km^3 of water brought by small wadies from Iran, which drain directly towards the marsh area. Euphrates River doesn't have tributaries inside Iraq. Groundwater resources are estimated about 1.2 BCM and form about 2% of the total water resources of Iraq (World Bank, 2006).

Iraq started controlling its water resources since 1939 when the first barrage was constructed at Kut on Tigris River. The idea of building dams in Iraq started in the first half of the twentieth century. Primarily it was to protect Baghdad the capital and other major cities from flooding (Fig. 1). The first big dam (Dokan) was constructed on the Lesser Zab River. They started its construction in 1956. Later, dams and regulators were constructed for irrigation and power generation purposes (Iraqi General Commission for Dams and Reservoirs, 2006; Iraqi Parliament, 2009). The natural depressions are included within the hydrological scheme as flooding escapes such as Al-Therthar depression.

The Iraqi Government realized the process of building dams should be speeded up due the huge increase of water demand and the threat of halting water of the rivers by Turkey and Syria. The process was stopped in the 1990s due to the second Gulf War and UN sanctions. None of these dams was filled to its maximum storage capacity during the twenty first century. This is attributed to the depletion of flow in the Euphrates and Tigris Rivers due to the Turkish and Syrian dams. It is noteworthy to mention that Haditha Dam is almost of no use now due to the severe depletion of the Euphrates flow (Al-Ansari, 2013; Al-Ansari and Knutsson 2011; Al-Ansari et al., 2014 and 2015).

The idea of building of Mosul Dam project started in 1950 and it was referred to as Aski Mosul Dam (Iraqi Ministry of Water Resources, 2014). The location of the dam was suggested by two British companies Alxeander Ghbbs and partners and Munsell Bassford and Bafery in 1953 to be at a village called "Dhaw Al-kamar" which is located 12 km north of Aski Mosul. The dam was designed so that its capacity reaches 8.7 km^3 at 320 m (a.s.l.) while the maximum elevation of the dam reaches 324 m (a.s.l.).

Later in 1956, the Iraqi Development Council asked an American company (Koljian) to carry out a new site investigation for the dam to use it for irrigation purposes. This company handed its report in 1957 suggesting that the dam should irrigate about 2500 km^2 distributed at North, South and East Aljazera. In the same year, the Iraqi Government asked Harza Company to perform a new site survey and design for the dam. In 1960, Harza Company suggested two sites for the dam different from those suggested earlier by other companies. This is attributed to the fact that,

earlier locations are not suitable because the dam will be built on highly soluble gypsum and very thin clay beds. The first suggested site was to build a dam with a storage capacity of 7.8 km^3 at 320 m (a.s.l.) and the other site was at 335 m (a.s.l.) with a storage capacity of 13.5 km^3. In 1962, the Iraqi Government asked Technoprom Export (Soviet company) to perform another investigation for the site of Mosul Dam and to suggest a new design as well. The company suggested a new site that is 600 m south of the site suggested by Harza Company. The dam was designed with a storage capacity of 7.7 km^3 at 312.4 m (a.s.l.) and the maximum height of the dam was supposed to be 83.7 m.

All the above companies suggested that the dam should be of rock-fill type with compressed clay core but there were different views about the exact location of the dam, spillway and electricity generating station. Grouting was suggested to be performed under the dam, spillway and the electricity generating station. In addition, they suggested that detailed geological investigation should be performed before any construction activities. In view of these reports, the Iraqi Government asked a Finish company "Imatran Voima" in 1965 to carry out new investigations. The dam at that time was supposed to irrigate 7500 km^2 at Mosul, Baghdad, Kut, Nasiriyah, Amarah and Basra Governorates. The company suggested a site which is located 60km northwest Mosul city. It was pointed out the geology of the area is so complex and requires further investigations. Another Yugoslav company (Geotcnica) worked on the geology of the suggested site in 1972 according to the advice of the International board of dams (IBD) and the report submitted by the Russian Technoprom Export Company. Imatran Voima Company carried out another investigation in 1973.According to these reports, IBD recommended in 1974 extra geological investigations.

The Iraqi Directorate General for Dams asked a French company (Soletanch previously known Soleseif) to perform more geological investigation on the suggested site. This was done during 1974-1978. Later in 1978, the Swiss Consultants Consortium was asked to be the consultants for Mosul Dam project. The consultants suggested that the operational water level at the dam to be 330 m (a.s.l.) while the flood and normal water levels to be 338 and 335 m (a.s.l.), respectively. A consortium of German and Italian companies (GIMOD) was asked to execute the civil and steel work of the project in 1980. While electromechanical plant of the power station was given to the Japanese company (Toshiba) on the condition that the capacity of the plant will be 750 MW. The electromechanical plant contract for the regulation dam was awarded to (Elin Union) from Austria, while the electromechanical plant for the pump storage scheme was given to (G.I.E) from Italy. The work started on 25th January, 1981 and finished 24th July, 1986. The total cost of the development was estimated at 2.6 billion US$ at the prices level of 1985.

In this research, the project area will be discussed and problems encountered will be highlighted.

2. Climatic Features

The climate of the catchment area can be regarded as being similar to a Mediterranean climate except some differences due to the presence of a mountainous region which is located

within the Turkish territory. The climate is a hot-dry summer and cold-rainy winter with occasional snowing in the mountains region. The precipitation in the Tigris River basin occurs between October and May. The annual precipitation over the Tigris basin ranges between 450 - 1000 mm annually (Al-Ansari and Knutsson, 2011; Al-Ansari et al., 2014 and 2015) while it is 200-600 mm at the dam site (ECB, 2010). The heaviest precipitation occurs from December to February. Generally, snow melting begins in February. Therefore, the flood runoff continues to May or early June. After this the flow rates are reduced where the lower rates occur in August to October. During this period the main source of the river runoff is the groundwater. The average monthly temperatures range between 6°C in January to 34°C in July but the temperatures decrease towards the north (ECB, 2010).

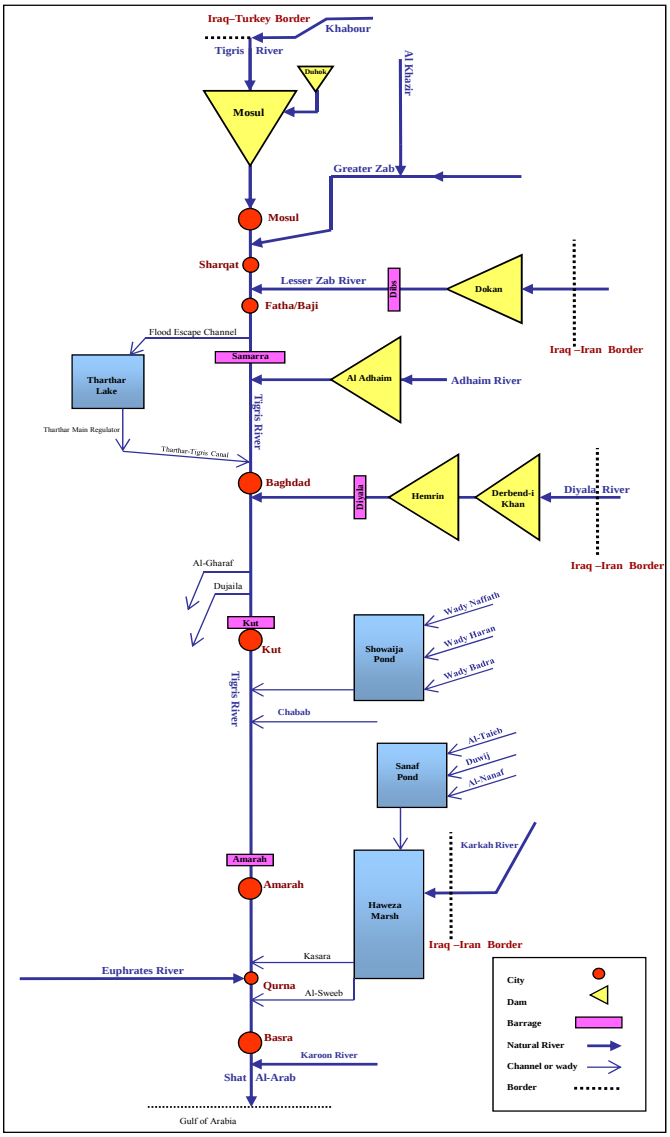

Figure 1: Schematic diagram of Tigris River and its tributaries hydrological scheme (IMWR, 2005).

3. River Tigris and Catchment Area of Mosul Dam

The Tigris River is one of the two most significant rivers in western Asia and main source of water for Mosul Dam reservoir. The main source for the Tigris River is Hazar Lake, which is

located in the south eastern region of Turkey. The lake is surrounded by the Taurus Mountain chain where the elevation reaches 3500 m. The catchment area of the River Tigris is divided geographically into three regions: mountainous, foot hills and the plain region. Its estimated that the catchment area upstream of Mosul Dam reservoir is about 54900 km^2, which is shared by Turkey, Syria and Iraq (Swiss Consultants, 1979, and Saleh, 2010) and the catchment area of the valleys surrounding the reservoir is about 1375 km^2 (Ezz-Aldeen et al., 2012). The Tigris River flows in hilly regions located to the south western part of the mountainous area connecting Turkey, Iran and Iraq. The River crosses the Iraqi border in Faish Khabur village which is located about 400 km from the main source and 128 km upstream of Mosul Dam. Four major tributaries, Batman, Garzan, Botan and Al-Khabur feed the Tigris River north of Mosul Dam from the left bank (Najib, 1980; Al-Ansari and Knutsson, 2011). Six large dams in Turkish territory had been constructed on the River Tigris upstream Mosul Dam during the last century (UNEP, 2001). The channel of the Tigris River is shallow and wide in the Diyarbakir area, but after it merges with the Batman tributary it becomes a narrow and deep river with high velocity. The width of the river valley (flood plain) north of Mosul city to Faish Khabur before Mosul Dam construction ranged from 2 to 10 km and the average water surface slope in this reach was 0.65 $m.km^{-1}$ (Swiss Consultants, 1979; Najib, 1980). The banks of the river valley have steep slopes from the right hand side and gentle low slopes from the left hand side. The most significant features of the River Tigris basin are given in (Table 1).

Table 1: Characteristics of the Tigris Rivers basin.

Tigris River	Turkey	Iraq	Syria	Iran	Total
Discharge $km^3.yr^{-1}$	33.5	6.8	0.0	11.2	51.5
Discharge (%)	65.0	13.2	0.0	21.8	100
Drainage Area km^2	45 000	292 000	1 000	37 000	375 000
Drainage Area (%)	12.0	54.0	0.2	33.80	100
River Length (km)	400	1318	44	—	1862
River Length (%)	21.0	77.0	2.0	—	100

Source: [UNEP, 2001; Biedler, 2004].

The annual hydrograph for the Tigris River starts from October to September. The highest mean monthly discharge takes place during April and the driest month is generally September (Fig. 2).

The average monthly discharge for the River Tigris is 631 $m^3.sec^{-1}$ for years 1931 to 2013 and the maximum discharge was 3514 $m^3.sec^{-1}$ in April 1954 while, the minimum was 81 $m^3.sec^{-1}$ in October 2013 (Fig. 3). The sediment on the bed of the river before construction of the dam had a median grain size diameter of d_{50}=18 mm (Swiss Consultants, 1979; Najib, 1980). In 2009, the sediments of the river were studied by the Dijla Company for Engineering Design and they noted that the specific gravity for bed material was Gs= 2.65 while the median grain size diameter of the sediment was d_{50} = 12.4 mm (ECB, 2010).

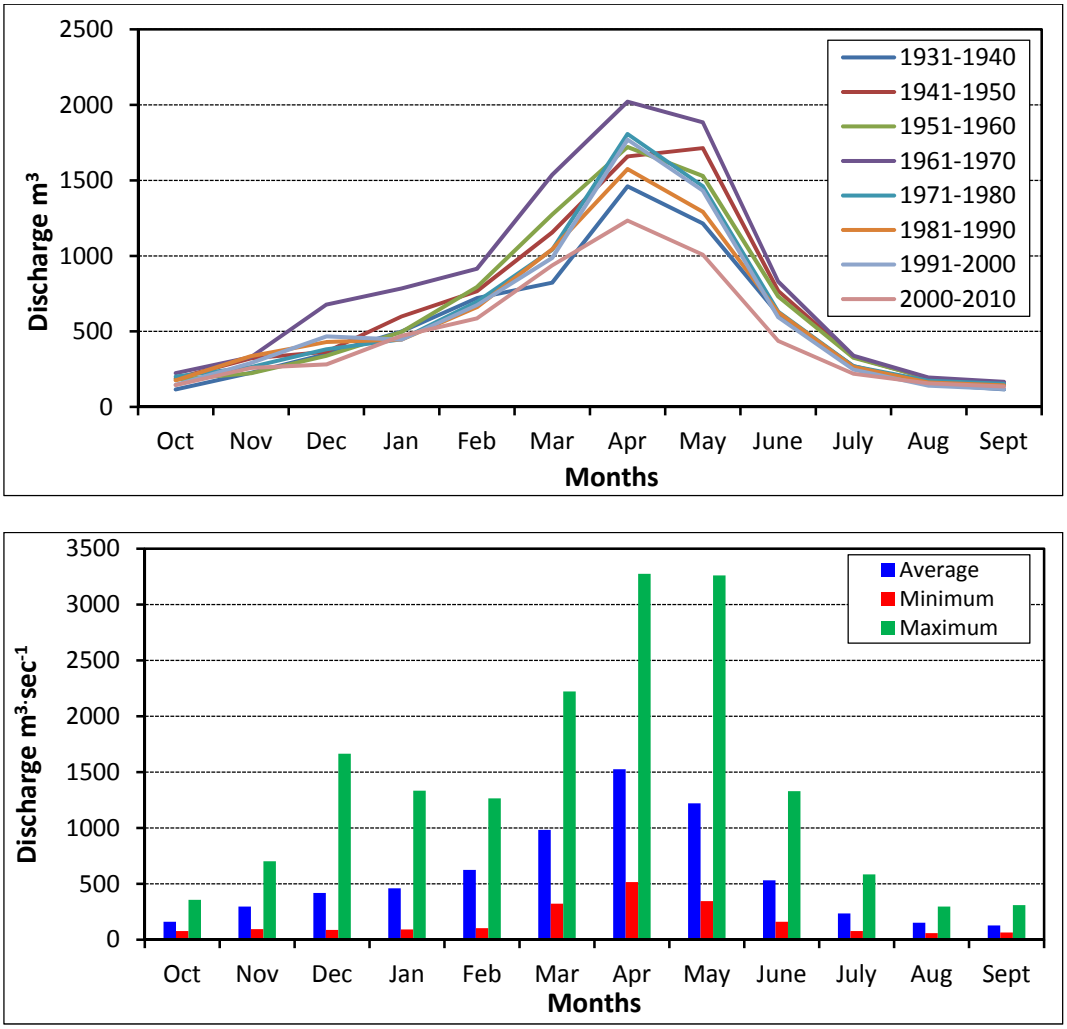

Figure 2: Monthly (mean, minimum and maximum) inflows of Tigris River at dam site (1931-2013).

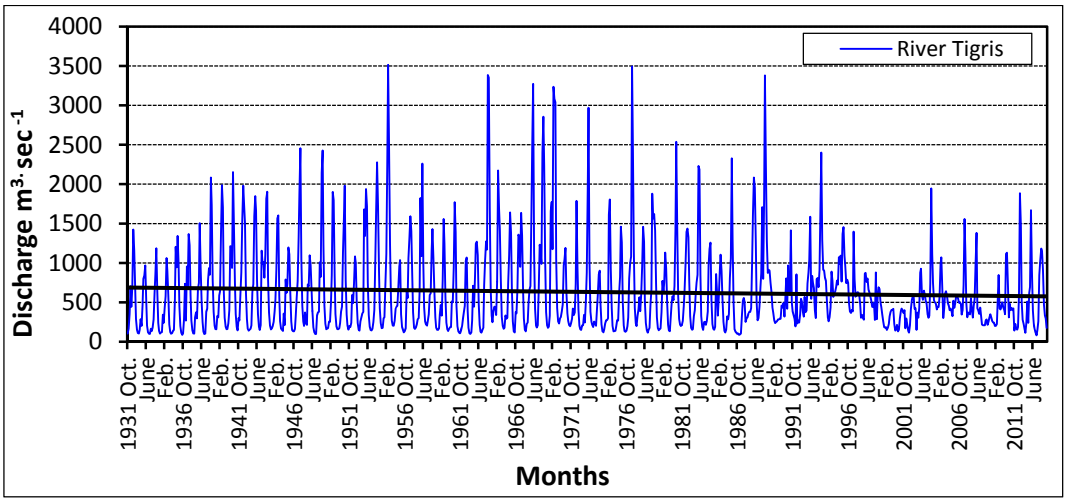

Figure 3: Average monthly inflow and its trend line of the Tigris River at dam site (1931-2013).

4. Mosul Dam

Mosul Dam is one of the most important strategic projects in Iraq for the management of its water resources. The project was constructed on the Tigris River in the north of Iraq, located 60 km northwest of Mosul city and 80 km from the Syrian and Turkish borders at 4056066 m N northing and 305356.69 m E easting (Iraqi Ministry of Water Resources 2012) (Fig. 4). Construction of Mosul Dam began on January 25th, 1981. The dam is a multipurpose project and it started operating on 24th July, 1986, to provide water for three irrigation projects at the north of Iraq and other projects in the middle and south of the country, flood control and hydropower generation. The dam is 113 m high, 3650 m long including the spillway, has a 10 m top width and the crest level is 341m (a.s.l.). The dam is faced with rock and has an earth fill with a clay core (Iraqi Ministry of Water Resources 2012). The maximum, normal and dead storage levels of the reservoir are 335, 330 and 300 m (a.s.l.) respectively. The dam was designed to impound 11.11 km^3 of water at normal operation level, including 8.16 and 2.95 km^3 of live storage and dead storage respectively (Fig. 4 and 5).

The dam has a concrte spillway located on the left abutment of the main dam (Fig. 4). The crest elevation of the spillway is 330 m (a.s.l.) and its length is 680 m. The spillway has five radial gates; measuring 13.5 m×13.5 m giving a discharge of 12600 $m^3.sec^{-1}$ at the maximum reservoir level of 338 m (a.s.l.) (Iraqi Ministry of Water Resources 2012).

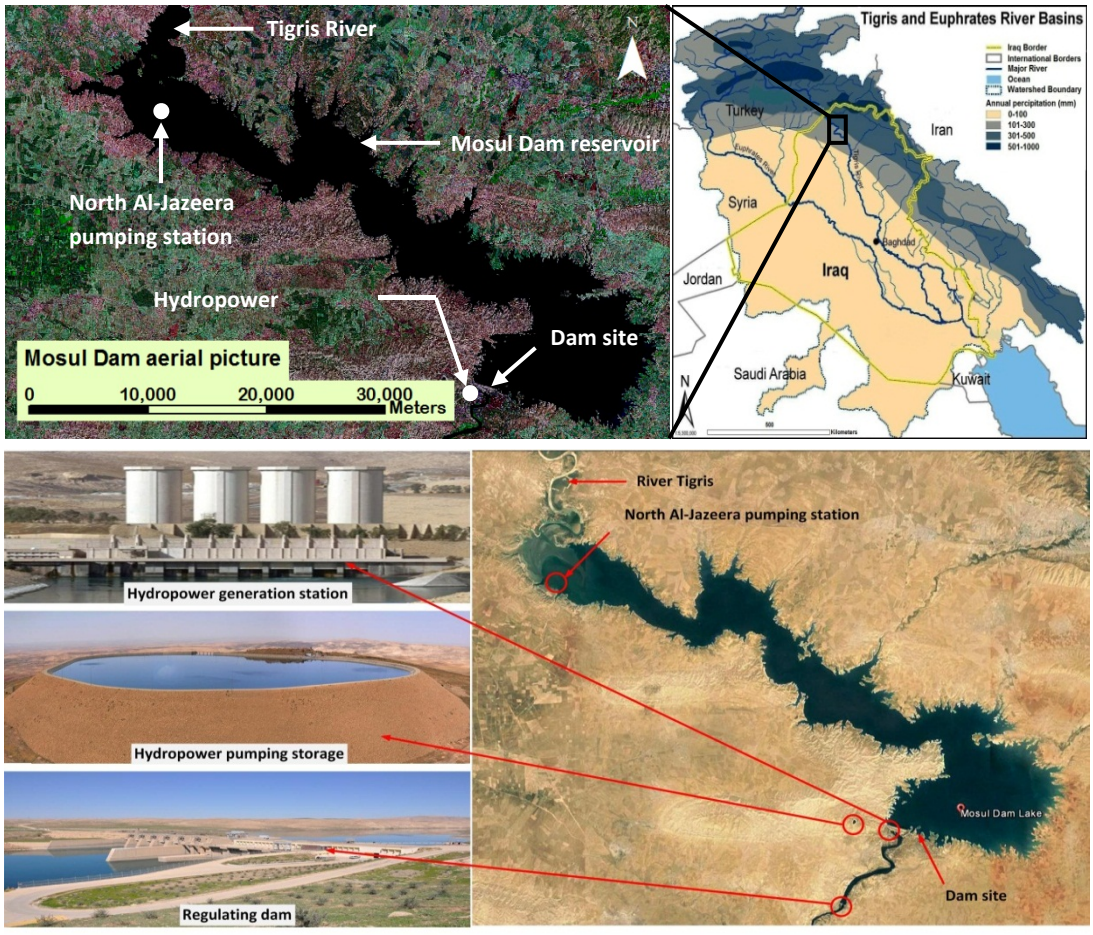

Figure 4: Location of Mosul Dam with main facilities.

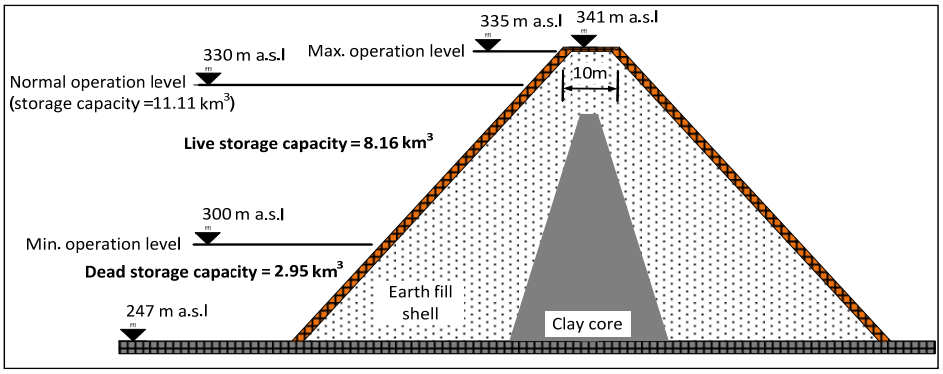

Figure 5: Schematic diagram of Mosul Dam cross section.

The power generation and pumping station of the north Al-Jazeera project is the important structures within the dam project, which were relied upon to monitor the water level during the bathymetric survey. The power generation facilities are located on and in the right abutment of the main dam (Fig. 4). The power house is located in the toe of the dam embankment and includes four turbines with total generating capacity of 750 MW. The Al-Jazeera pumping station is located in the upper zone of the reservoir 278409 m E and 407663 m N with a maximum water discharge 45 $m^3.sec^{-1}$ (Fig. 4) (Iraqi Ministry of Water Resources 2012).

5. Mosul Dam Reservoir (MDR)

The reservoir is located between latitude (4055000 to 4086000) m N and longitude (275000 to 320000) m E. The shape of the reservoir is almost elongated where the River Tigris enters the upper zone and expands close to the dam site. The length of the reservoir is about 45 km and its width ranges from 2 to 14 km with a water surface area of about 380 km^2 at the maximum operation level of 330 m (a.s.l.). There are seven main valleys that feed the reservoir from the left side and three from the right side of the reservoir (Ezz-Aldeen, et al., 2012). The characteristics of these valleys are shown in (Table 2.2). The soils of these valleys are mostly silty loam, silty clay, loam and clay. On the other hand the annual sediment delivered by right and left valleys of MDR are 42.7×10^3 $ton.yr^{-1}$ and 702×10^3 $ton.yr^{-1}$, respectively (Ezz-Aldeen, et al., 2012; 2013).

Table 2 : Properties of the main tributary valleys around Mosul reservoir.

Valley name	Side feeding	Area (km^2)	Slope	Length (km)	Mean basin level m (a.s.l.)
Sweedy	Right	450.76	0.0359	38.8	446.62
Kara Kandy	Right	78.52	0.0217	21.82	388.38
Khuyr Hara	Right	50.06	0.0525	10.86	404.89
Amlik	Left	88.95	0.0281	38.94	470.42
Jardyam	Left	88.73	0.0215	52.68	457.1
Affkery	Left	139.5	0.0214	58.04	445.34
Khrab Malk	Left	119.6	0.0255	51.32	475.87
Naqeb	Left	104.1	0.0143	54.71	426.52
Kalaq	Left	162.26	0.0173	60.52	424
Saeed Thaher	Left	92.25	0.026	43.23	414

Using the data available from the Iraqi Ministry of Water Resources the average monthly inflow and outflow of the reservoir was 561 and 555 $m^3.sec^{-1}$ for the period 1986 to 2011of its operation (Fig. 2.5). Mosul Dam provided water for three irrigation projects, power generation, regulation and flood control. Dam operation started during June 1984, with initial reservoir filling during the spring of 1985 but the actual operation began in July, 1986 (Iraqi Ministry of Water Resources, 2012). The operation mode of the dam during 1986-2011 is shown in figures (2.5) and (2.6) for discharges and water elevations respectively.

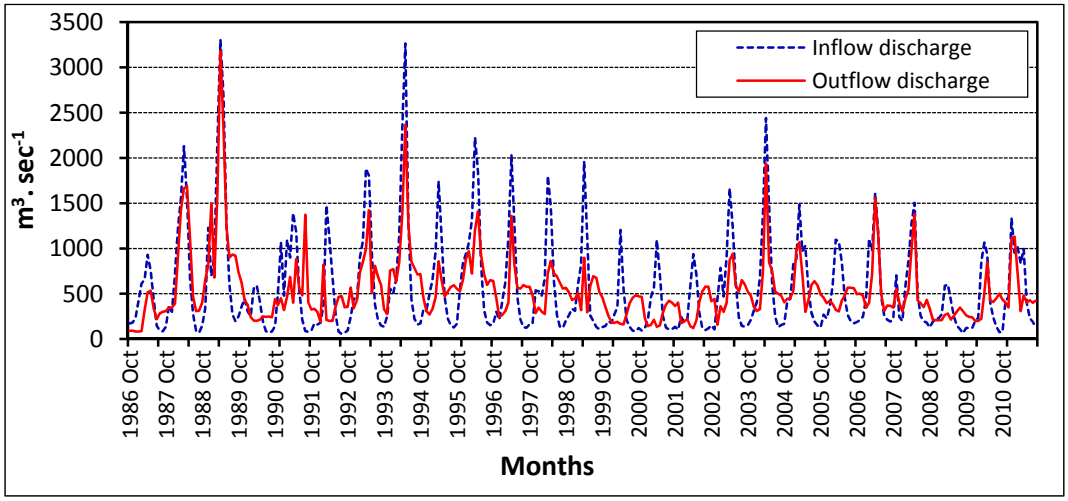

Figure 6: Average monthly inflow and outflow discharges of MDR for 1986-2011.

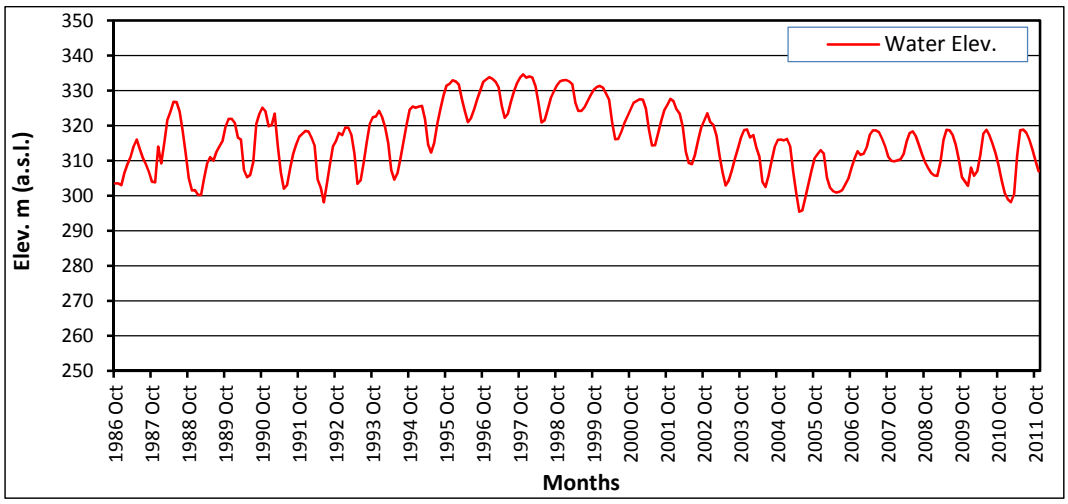

Figure 7: Average monthly water elevations of MDR for 1986-2011.

6. Geology of the Dam Site

The dam abutments are located on the Upper Member of the Fatha Formation (Middle Miocene) (Fig. 8). The Upper Member, as the Lower Member of the Fatha Formation consists of cyclic sediments, marls, claystone, limestone and gypsum; however, in the uppermost part the claystone ratio increases as compared with the lower part. These inhomogeneous rocks; in their mechanical behavior will certainly behave differently when are loaded. The gypsum and limestone beds are usually karstified, but the karstification is less in the Upper Member as compared to Lower Member of the Fatha Formation (Sissakian, 1978; Sissakian and Al-Musawi,

2007). This is attributed to the presence of more clastics in the Upper Member than that of the Lower Member of the Fatha Formation, besides that gypsum and limestone beds become thin in the uppermost parts of the formation. Figure 9 show the detailed description of the beds at Mosul Dam site.

Due to the complexity of the geology of the area, several investigations were carried out (e.g. Al-Ansari et al., 1984; Wheeler et al., 2005; Wakeley et al., 2007). Al-Ansari et al. (1984) reported that there were two faults within the dam site area. The fault is of rotational type striking NW-SE offsetting the northern corner of Butmah East structure. The throw of the fault increases towards the east. The fault plane is believed to be nearly vertical near the surface with a slight dip towards the north. They also reported that this fault extends to the dam site. The other fault is trending N NE-S SW along the right bank of the River Tigris. This fault is not clear on the surface and it was detected using Landsat images. This is due to the fact that the fault is deep seated and it might be partly controlling the Tigris River course from the dam site to Aski Mosul. Wakeley et al. (2007) collected all the borehole data and constructed a model to show the complex geology at the dam site (see figures 10 and 11).

No nontectonic activity is reported from the dam site and near surroundings (Sissakianet et al., 2011), as well the present small faults, as discovered in the dam site during execution of detailed geological mapping by Iraq Geological Survey (Hagopian, 1984, personal communication) has no significant importance and/ or cause any hazard for the dam. Even the main deep seated Sassan – Be'Khair Fault (Fouad, 2012) shows no surface indication for any recent activity, as recognized by Saleh (2004). Therefore, the surface area of the dam site almost doesn't suffer from active tectonic disturbances; this is also confirmed by Kelly et al. (2007), when they stated that the dam body does not suffer from any problem.

The most significant geological hazard that influences Mosul Dam is the karstification, especially in the foundation's rocks. According to Sissakian and Ibrahim (2004) the karstification had scored 5.5 degrees within the existing geological hazards; not only in the dam site but, in the whole Mosul Quadrangle at scale of 1:250 000 that covers an area of about 30000 Km^2. The influence of the karstification is also confirmed by Swiss Consultants (1979), Al-Ansari et al. (1984), Black and Veatch (1984), Mark and Wheeler (2004), Sissakian and Ibrahim (2004), Washington International Group (2005), Hijab and Al-Jabbar (2006), Sissakian and Al-Musawi (2007), Hijab et al. (2007), and Sissakian et al. (2014). Another geological hazard in Mosul Dam and reservoir is the slope stability. The slope stability of dams is discussed and confirmed by many authors, among them are: Lane and Griffiths (2000), Sissakian et al. (2003 and 2005), Xu et al. (2009) and Keming and Bagale (2012). The landslides occur either due to quick down draw or oversaturation of the rocks, especially when claystone or marl occur in the rock sequence surrounding the reservoir. In Mosul, the lithology of the Fatha Formation is very favorable for development of landslides, especially when the beds are dipping towards the reservoir, which is the case in Butma East, Ain Zala, Musoura, Dahqan and Qand anticlines. However, the existing landslides in the reservoir area are not so large; therefore, they did not impose significant hazard on the dam body.

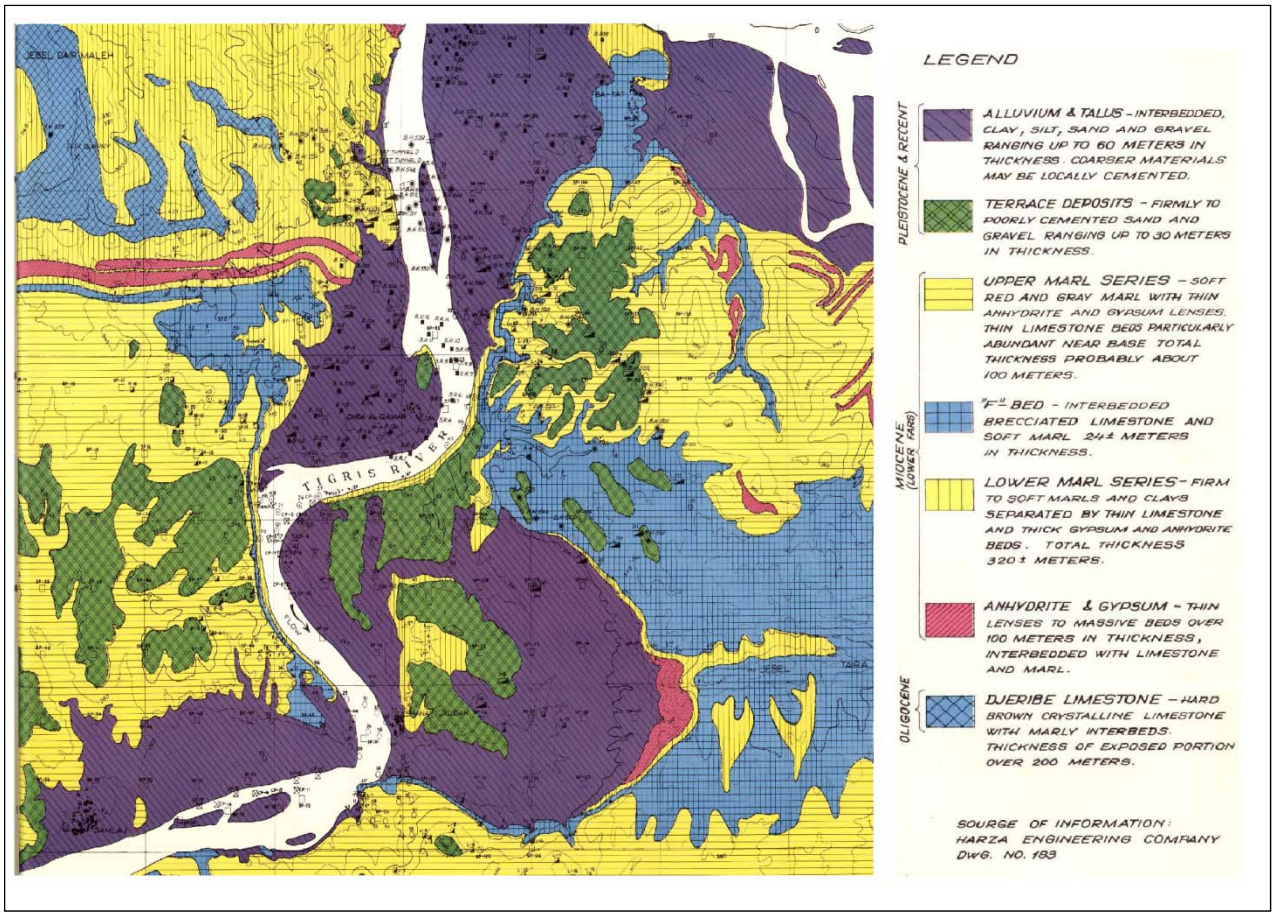

Figure 8: Geologic map of Mosul Dam site area (After IVO, 1969).

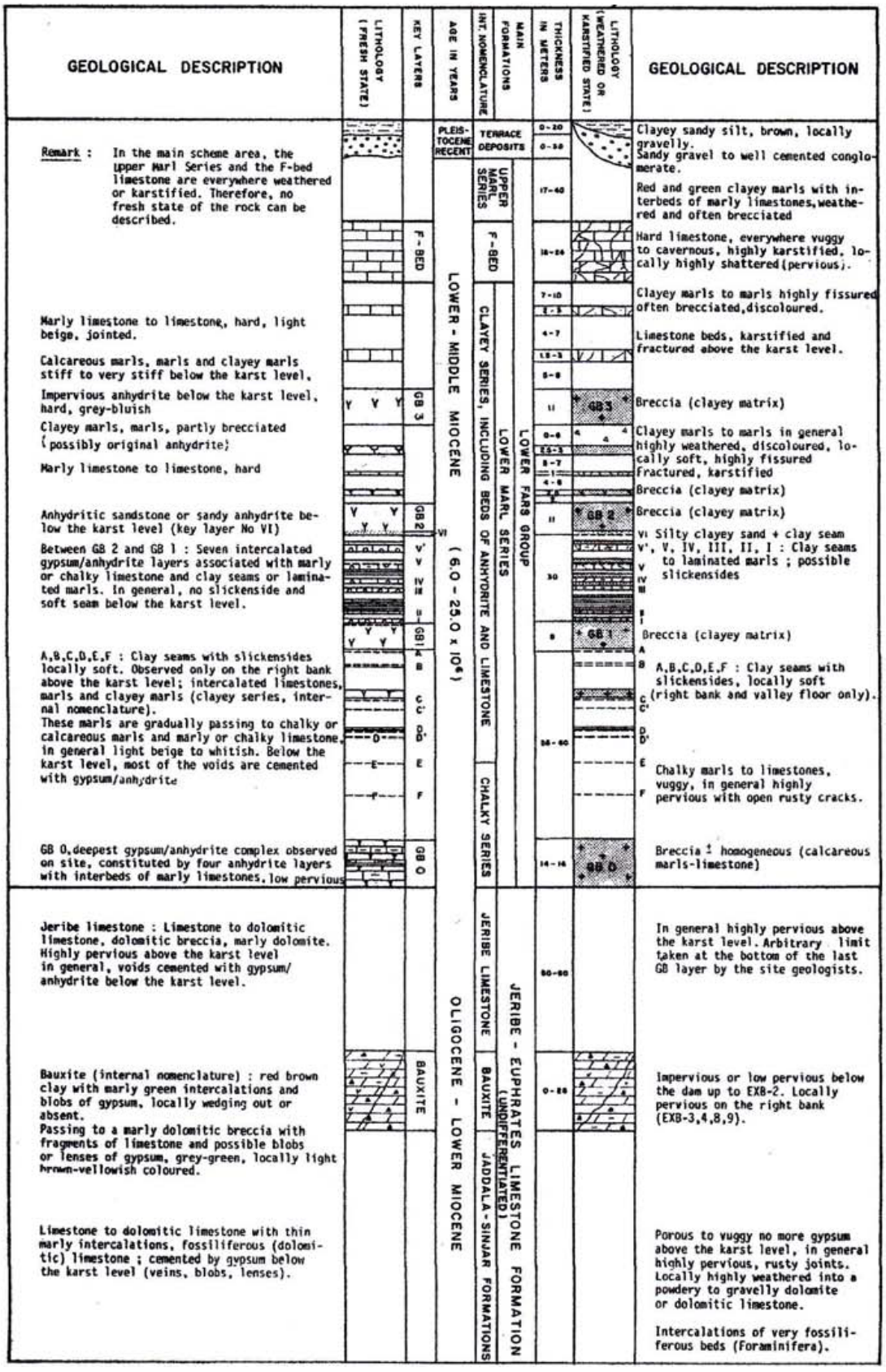

Figure 9: Lithological column of beds at Mosul Dam site (After Wheeler et al., 5005).

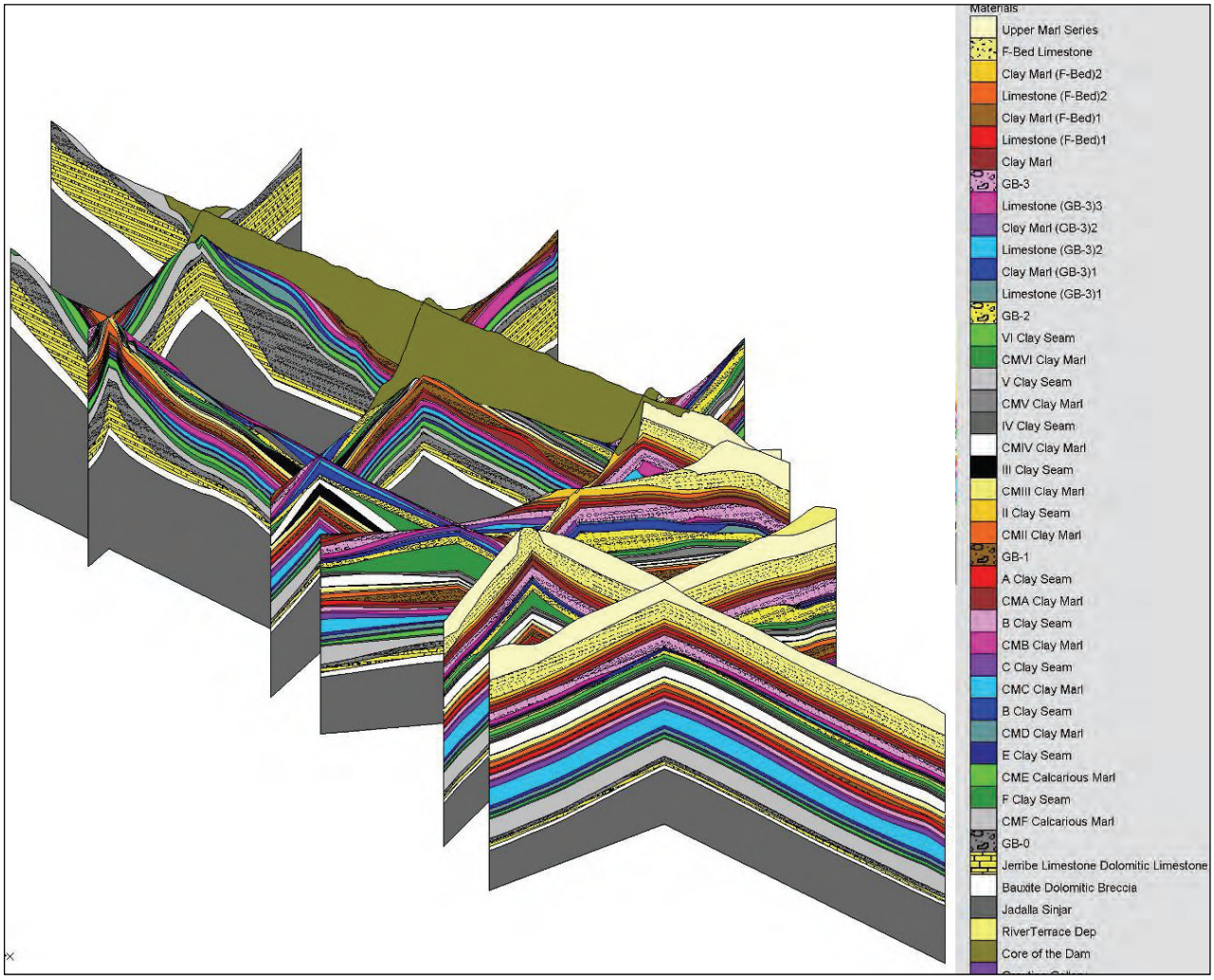

Figure 10: Intersecting cross sections from intermediate version of ERDC geologic conceptual model, showing complex stratigraphy and partial resolution of discrepancies in stratigraphy at intersections of the geologic panels from generated boreholes. (Kelly *et al*., 2007).

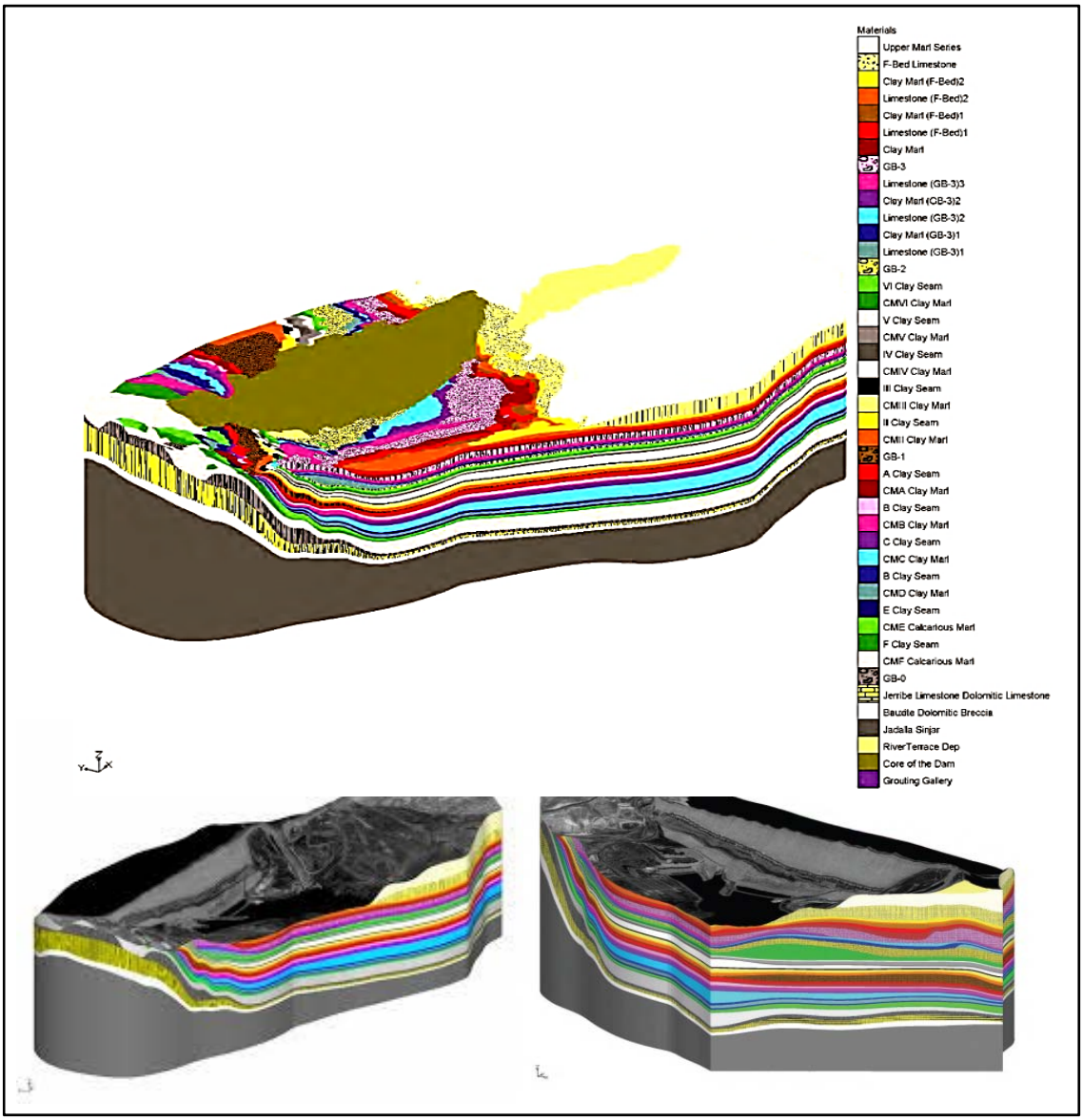

Figure 11: Three dimensional model of Mosul Dam site (Kelly *et al.* ,2007).

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CHAPTER 3: KARSTIFICATION AND SINKHOLES

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1. Introduction

Karst topography forms due to the dissolution of soluble rocks like gypsum, and limestone and less common dolomite. It is usually characterized by underground system of dolines, caves and sinkholes. This phenomenon is usually associated with different features. They might be large features like pavements, poljes, and karst valleys, or medium size features like sinkholes or closed basins (cenotes), vertical shafts, inverted funnels shaped sinkholes (foibe) and small size features like flutes, runnels, clints and grikes, collectively called karren or lapiez. Internal drainage, subsidence, and collapse triggered by the development of underlying caves are the processes that form the surface karst features (Palmer, 1991). Rainwater becomes acidic in contact with carbon dioxide in the atmosphere and in the soil. When water infiltrates in rocks it will start to dissolve away the rocks. This will create a network of passages. Over time, water flowing through such network continues to dissolve and enlarge the passages; this will allow the plumbing system to transport larger amounts of water (Gunn, 2004). Sinkhole is a hole or depression in the ground formed by the collapse of the surface layer or by the solution process. They vary in size, depth and shape. They are more common in areas where the rocks below the land surface are limestones or other carbonate rocks, salt beds, or gypsum, that can be dissolved naturally by circulating ground water.

Sinkholes are either active or inactive. Those, which are still active have one or more outlet in their floors, which extend into shallow funnel shaped caves (Sissakian et al., 1986; Al-Naqash and Al-Talabani, 1988) this means that the active sinkholes have the ability to drain the infilling water to deeper horizons, or ground water runs through them. These types of sinkholes are certainly more problematic. The indications of the activity are (Sissakian et al., 1986):

- bare floor or with very rare soil cover
- presence of one or more outlet in the floor
- presence of fallen rock blocks from the rim in the floor
- presence of circular or crescent-shaped cracks around the rim
- presence of ground water in the floor

Those sinkholes, which are inactive, are less problematic, because they exhibit less deformation to the near surroundings. This is attributed to the fact that they will not be able to transfer water into deep horizons, or at least the transferred amount is lesser; if compared with those of active sinkholes with same size and same conditions. The inactivity of the sinkholes is indicated by (Sissakian et al., 1986):

- spoon shape
- cover of thick soil in the floor
- absence of fallen blocks from the walls and /or the rims in the floor
- absence of outlets in the floor
- presence of outlets in the rim
- presence of water accumulation after heavy rain showers, in the floor
- presence of vegetation in the floor during rainy seasons
- absences of cracks around the rims

One of the main reasons of the karstification is dissolving of the limestone by carbonic acid, which is formed by reaction of the water with carbon dioxide. However, gypsum is dissolved by sulfuric acid, which is formed by reaction of the oxygen with H_2S. As oxygen (O_2)-rich surface waters seep into deep anoxic karst systems, it brings oxygen which reacts with sulfide present (H_2S) in the system to form sulfuric acid (H_2SO_4). Sulfuric acid then reacts with calcium carbonate causing increasing erosion within the limestone formation. This chain of reactions is:

$H_2S + 2\ O_2 \rightarrow H_2SO_4$ (sulfide oxidation)
$H_2SO4 + 2\ H_2O.\ SO4 - 2 + 2H_3O+$ (sulfuric acid dissociation) (sulfuric acid issociation)
$CaCO_3 + 2\ H_3O^+ \rightarrow Ca^{2+} + H_2CO_3 + 2\ H_2O$ (calcium carbonate dissolution)
$CaCO_3 + H_2SO_4 \rightarrow CaSO_4 + H_2CO_3$ (global reaction leading to calcium sulfate)
$CaSO_4 + 2\ H_2O \rightarrow CaSO_4 \cdot 2\ H_2O$ (hydration and gypsum formation)

It is worth mentioning that the Fatha Formation is an excellent source for H_2S, which is emitted due to the presence of native sulfur; almost everywhere in the formation. The area within the vicinity of Mosul Dam (Fig. 1) is characterized by its karst topography and the presence and appearance of sinkholes. In this section these phenomenon will be discussed in Mosul Dam area.

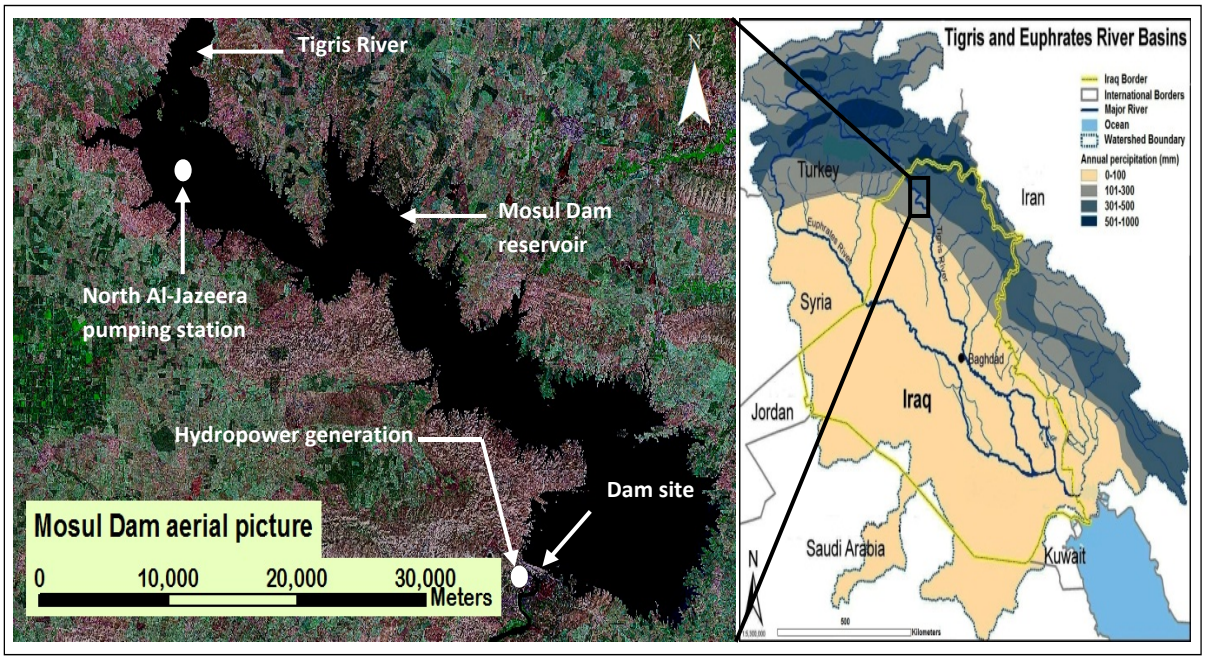

Figure 1: Location of Mosul Dam with main facilities.

2. Geology of the Dam Site

The Fatha Formation (ex-Lower Fars) (Middle Miocene) is the predominant stratigraphic unit in the Mosul Dam area. It is about 250 meters thick near Mosul (Jassim et al., 1997). Marl, chalky limestone, gypsum, anhydrite, and limestone form a layered sequence of rocks under the foundation of the dam. The foundation of the dam is mainly resting on Fatha Formation (Middle Miocene).The foundations of the dam are located on the Lower Member of the Fatha Formation and the Jeribe Formation (Black and Veatch, 2004 and Wheeler, 2004), although the Jeribe Formation was not recognized during the regional geological survey of the involved area and instead the Euphrates Formation was mapped (Taufiq and Domas, 1977). Lithologically, there is

not a difference between the Jeribe and Euphrates formations, but no karstification is reported in the exposed rocks of the Jeribe Formation, whereas the Euphrates Formation is highly karstified in different parts of Iraq, among them is Haditha vicinity near Hadith Dam site (Sissakian et al., 1984, 1986 and 2011; Jassim et al., 1987).

The right (west) abutment is located in the steeply dipping Fatha Formation within Butmah East anticline with SE plunge being in the reservoir north of the dam, whereas the left (east) abutment is located on gently dipping beds of the Fatha Formation, which is overlain by fine clastics of the Injana Formation. These differences in lithology as well the dip amount and direction along both abutments as well upstream and downstream of the dam have certainly affected on the hydraulic pressure and increased the dissolution ability of the gypsum and limestone beds, along the abutments and the foundations, which are already karstified in nearby areas. Consequently, more gypsum, anhydrite and limestone were dissolved and karst openings are continuously increased, as the exerted hydraulic pressure is continuous. Figure 2 show the detailed description of the beds at Mosul Dam site.

It should be mentioned however, that karstic limestone and the development of solution cavities within the gypsum and anhydrite layers are the main geological features under the foundation of the dam. The bore holes data indicated that four significant gypsum units were identified during design and construction varying in thickness from 8 to 16m and identified as GB0 (Gypsum Breccia 0), GB1, GB2, and GB3 in ascending order (Fig.3) (Wheeler et al., 2005). GB3 appeared at the excavation of the spillway foundations. It should be mentioned however, that the term gypsum breccia was used by Wheeler et al. (2005) and not the authors who prefer the term gypsum nodules.

The development of voids requires continuous grouting which is mainly due to the dissolution and erosion of gypsum by water seeping under the dam. Wheeler et al. (2005) believes that the erosion and dissolution rates in gypsum are related to the seepage velocities and hydraulic gradient. When calcium sulfate saturation is lower than approximately 2,000 ppm in seepage water, gypsum dissolution continues this zone will move downstream as greater quantities of unsaturated water attack a gypsum bed.

At Mosul Dam site, major dissolution occurs at the “karstic line”, where anhydrite converts to gypsum and this unit is subsequently dissolved and eroded by seepage (Fig. 4). This phenomenon takes place in Mosul Dam where evaporites are dissolved by groundwater and the voids are generally filled with collapse breccia from the overlying beds (Fig. 5) (Warren 2006).

3. KARSTIFICATION

The presence of rocks with high dissolution ability; such as limestone and gypsum in the abutments and foundations of dams, will certainly form karstification phenomenon, which will be increased with time, especially when karstification factors are available. Dissolution intensity at Mosul Dam ranged from 42 to 80 tons per day. This process coupled with the karstified limestone, dolomite, and marl as well as the evaporate rocks present the unfavorable foundation

conditions under Mosul Dam. The karst line also denotes the transition from the interbedded limestone-anhydrite/ gypsum beds to the less permeable Jeribe limestone (Bruce, 2005).

First appearance of the sinkholes on the right bank down-stream was not until approximately six years after beginning of the reservoir filling (Fig. 6). The surface expression of the sinkholes suggests that they are caused by an under-ground collapse. Concentric tension cracks appear to have developed around the central void as the sinkholes have developed progressively (Wheeler, 2004).

The majority of karst forms are sinkholes (Fig. 6) (dolines), developed in gypsum and/ or limestone; however, many other forms are developed too, such as Karren, shafts, channels. Downstream sinkholes are most likely related to fluctuations in the tail water level of the main dam during operation of the dam and the down-stream regulating reservoir (Washington Group International, 2005). These sinkholes may be connected to an aquifer on the right bank of the reservoir, because before the reservoir was impounded, substantial flows of order of 360 l/s were encountered from a gypsum layer found during excavation of the tailrace tunnel for the pumped storage scheme; the water had a high sulphate content, which was different from the reservoir water (Fig. 7). The karst line also denotes the transition from the interbedded lime-stone-anhydrite/gypsum beds to the less permeable Jeribe limestone (Fig. 4) (Bruce, 2005).

Karst development extends to a depth of about 100 m below the base of the dam. This may be a relict karst from a former climatic regime when groundwater levels were lower. The concerns in the limestone units are basically related to the existing caverns, voids, and fractures and not so much due to the erosion and dissolution process, as limestone generally dissolves at very slow rates. The main drawback to the site is the presence of soluble rocks by the presence of anhydrite and gypsum and the associated karst conditions at the foundation (Wheeler, 2004). Karst area has high permeability conduits that convey substantial quantities of water at varying velocities.

Surface cracking and ground settlement initially developed followed by appearance of sinkholes on the right abutment. One sinkhole on the left flank of the reservoir (in the local tourist village) appeared suddenly and developed rapidly with 15 m of settlement and a 15 m diameter depression; overnight in February 2003 (Figures 9 and 10). The sinkhole was dry, located on a slope on the northern side of a valley feature. The deposits at the surface standing vertically appear to be superficial silty soils, with perhaps bedded marls below. Although the initial sinkhole was filled with 1200 m^3 of loose sandy gravel material and fenced off, but the settlement continued. A further 3000 m^3 of material was required to fill the settled surface of the sinkhole in two separate filling operations in May 2003 and October 2004. By March 2005 there were been a smaller settlement of the ground surface of about 0.5 m.

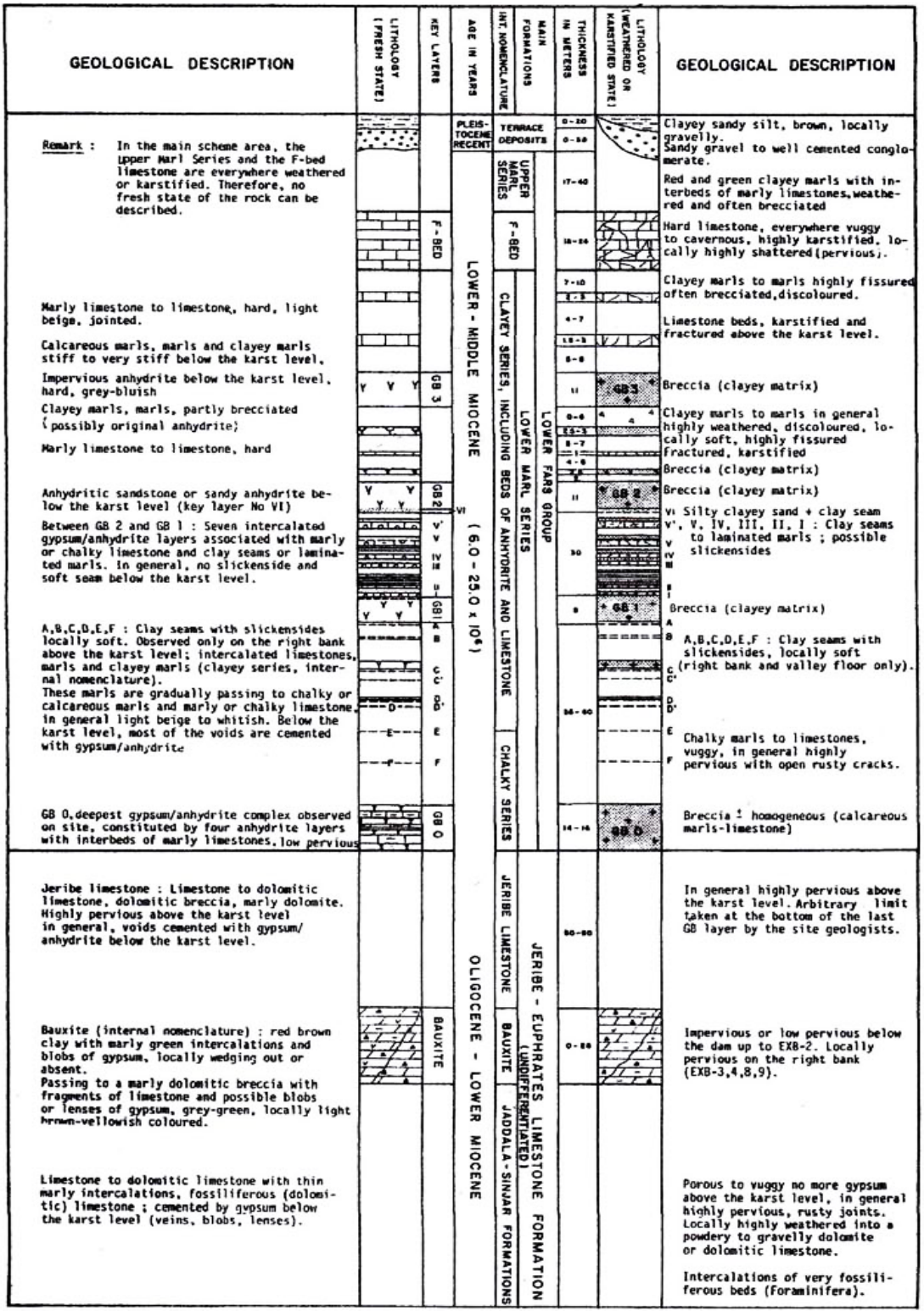

Figure. 2: Lithological column of beds at Mosul Dam site (after Wheeler et al., 2005).

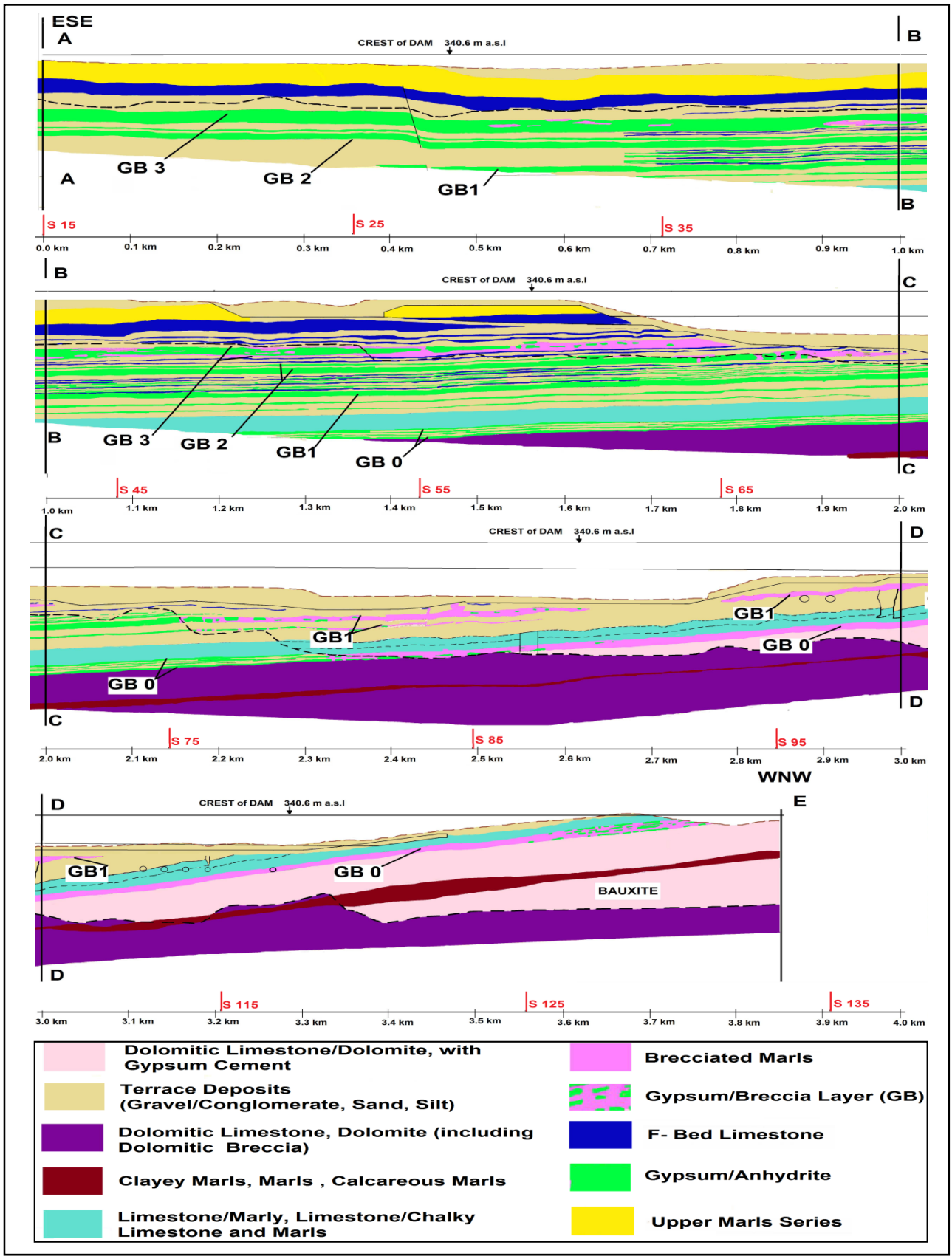

Figure 3: Geologic cross sections along the axis of the dam.

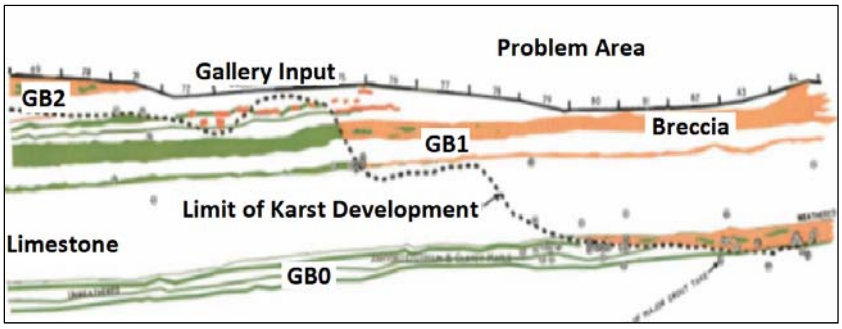

Figure 4: Location of the Karst line (Wheeler et al., 2005).

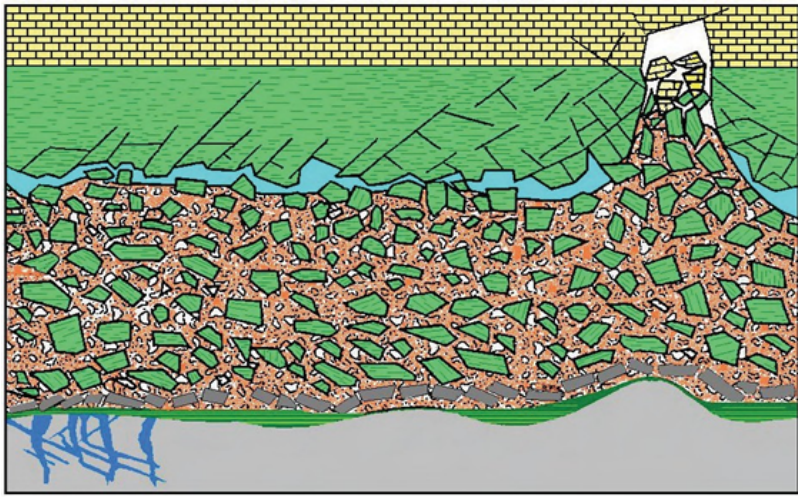

Figure 5: Development of breccia within a layer of gypsum (Warren, 2006).

Figure 6: One of the four sinkholes which had appeared in the 90's at the downstream right bank near the contractor yard.

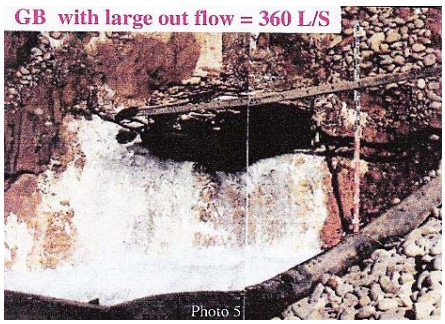

Figure 7: Spring of discharge 360 l/s which is connected to the ground water aquifer connected to the reservoir.

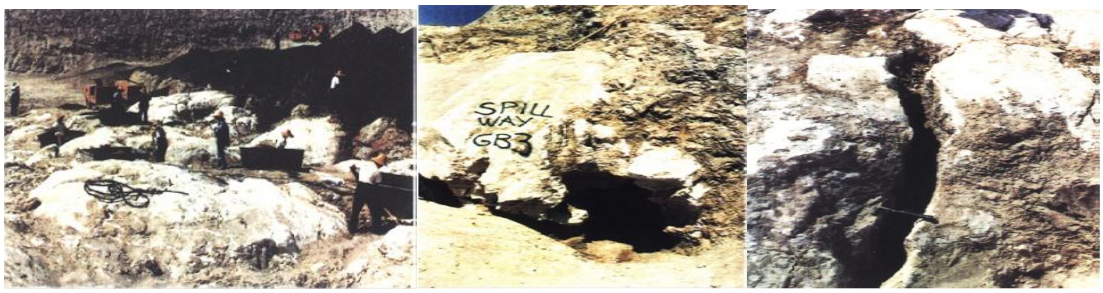

Figure 8: Karstified gypsum/anhydrite "GB3" exposed at the foundation excavations of the spillways foundations.

During the drilling of a piezometer hole near the sinkhole, a rod drop occurred. There was no water encountered in the hole so the piezometer was not installed, and the hole was grouted, taking 250 tons of grout. The suggestion was been made that the sudden appearance of the sinkhole is related to seepage water passing through the left abutment of the reservoir and the sinkhole lies close to the end of the left extension of the original grout curtain. Though possible, an increase in infiltration of surface water, perhaps from heavy rain, could cause a collapse into an existing void by destabilizing the ground (Washington Group International, 2005). Ground movement or collapse may have been initiated by changes in groundwater on impoundment of water in the regulating reservoir for those in the valley downstream of the main dam. The alignment of the sinkholes downstream of the dam strongly suggests a geological (structural) control and relatively near surface phenomenon. However, it has not been possible to verify this either by carrying out detailed geological field studies at the site or by obtaining high quality aerial photographs (Wheeler, 2004).

About 100 m up-stream from the dam, on the right abutment of the reservoir, a dipping limestone bed near the margin of the reservoir shows ground settlement. This movement could be partly related to sloping failure and sliding of the rock beds as a result of fluctuating reservoir levels and residual high pore pressures in the slope. However, a sinkhole feature seems more likely. Although tested for, no connection has been established between this upstream sinkhole and those downstream (Bruce, 2005). It is believed that in some cases soil failure takes place (Fig. 10), which could be initiated due to over saturation of the soil by rain water. In addition slab failure (Fig. 11) was also recognized, most probably due to fluctuation of the water level in the reservoir.

Figure 9: The left bank sinkhole after full development.

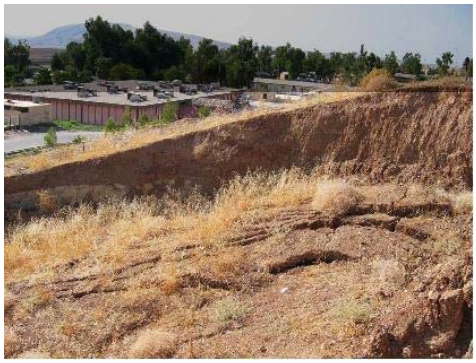

Figure10: The left bank sinkhole at early stage of development.

Figure 11: Upstream right abutment. Open fissure resulting from slab movement or sinkhole development underneath.

The prediction of sinkhole formation as well as the development of water bearing solution conduits beneath the embankment is obviously a critical issue. Previous surveys using echo sounding near the dam in the reservoir have reportedly not detected any evidence of sinkholes in the reservoir floor upstream of the dam. These surveys have recorded that sediment accumulation in the reservoir is limited to only about 1 m. This helps to blanket the floor of the reservoir and thus reduces inflow into the beds that might outcrop within the reservoir basin (Black and Veatch, 2004). Hijab et al. (2007), however, confirmed that some of the sinkholes are related to the reservoir. Their development is attributed to the development of underground conduits.

It is very clear that the karstification in Mosul Dam site is still active. It is also increasing in its activity causing a serious geological hazard to the status of the dam, if no relevant precautions are performed. This is also indicated from continuous grouting in the foundations of the dam. However, location of the sinkholes can be detected by means of geophysical studies (Hijab et al., 2007). Moreover, it is also possible to delineate the location of the subsurface channels (conduits) that are developed due to groundwater movement and dissolution of limestone and gypsum beds, which are the main rock types in Mosul Dam site.

Al-Ansari et al. (2013) and Issa et al. (2013) conducted a bathymetric survey during 2011 and concluded that *"After 25 years of the dam operation following; the thalweg bed slope of the River Tigris had changed from 0.65 m.km^{-1} before dam construction to 0.71 m.km^{-1}"*. The sedimentation rate in the upper section of the reservoir where the River Tigris enters the reservoir was greatest and gradually decreased toward the Mosul Dam site. The greatest deposition thickness was 17.6 m in the upper zone of the reservoir. Furthermore, there are many areas like the middle and lower parts of the reservoir that are exposed to erosion (Fig. 12). This is believed to be due to the dissolution of gypsum and limestone beds forming sinkholes that might reach about 20 m in diameter and 9.6 m in depth. The conducted survey also confirms the presence of sinkholes and other karst forms within the reservoir, which are active and continuously increasing in size, and exerting hazards to the Mosul Dam.

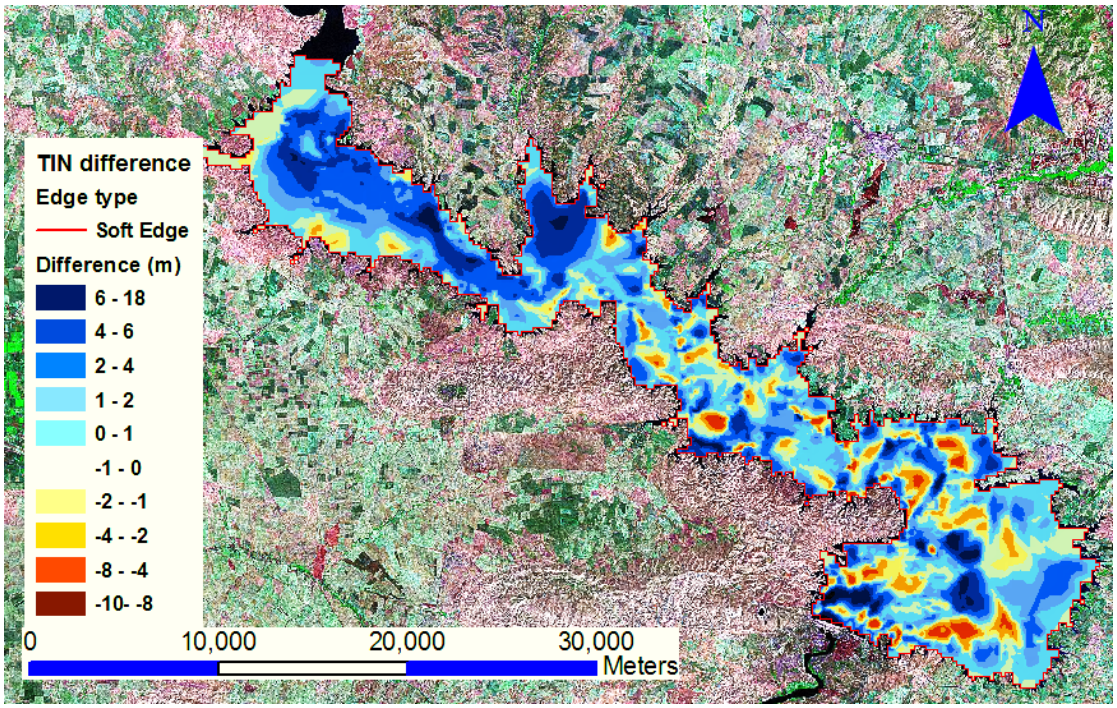

Figure 12: Holes noticed at the bed of Mosul reservoir (Issa et al., 2013).

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CHAPTER 4: PROBLEMS ENCOUNTERED DURING AND AFTER IMPOUNDING THE RESERVOIR

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2	Sinkholes Formation	49
3	Seepage and Dissolution of Gypsum	55
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1. Introduction

The first impounding of Mosul Dam reservoir started in 1986. The river section had been closed during the previous summer and the flow was diverted through the two diversion tunnels, while the dam section at the river channel was being built. By the end of 1985 the dam had been raised to such a height that the two diversion tunnels could be converted to the two gated bottom outlets and control could be exercised over the impounding operation.

At the end of the 1986 flood season the reservoir level reached 316.4 m (a.s.l.). The reservoir was filled to the normal operation water level of 330 m (a.s.l.) for the first time after the wet season of 1988 and this continued during most of the following years. The stored water was used during the dry seasons for irrigation and power generation as planned. This meant drawing down the reservoir level during the months from June through October each year before the start of the next wet season.

The new dynamics of the ground water which were created by the first impounding and then by the operation of the reservoir triggered and accelerated the sinkholes formation activity around the dam site, seepage and springs development downstream the dam and the deterioration of the deep grout curtain under the dam was observed. This curtain was already suffering from deficiencies during construction due to the inability of closing certain areas in the curtain within the gypsum brecciated layers. The open gaps in the curtain were called “windows”.

2. Sinkholes Formation

In September 1986, an inspection of the reservoir rims was carried out when water level had been drawn to EL. 309 m (a.s.l.) from EL. 316.4 m (a.s.l.), which had reached during the previous flood season. The inspection revealed the development of serious solution channels and sinkholes at the right bank at many points at about 150 meters from the contact with the right abutment of the dam. One sinkhole was also observed at about 1 km or so away. These solution channels showed dramatic dissolution of the gypsum layers which were exposed on the shore line (IBOE, 1987). Observations and further studies gave two possibilities for the future development of these sinkholes and underground solution channels. The first possibility is that they could progress in a direction parallel to the dam axis in this case there would be no adverse effect on the dam, or the second possibility is the development might cross the dam axis. In such event the right abutment of the dam would be endangered. In view of the uncertainty and to avoid any possibility of failure an intensive program of grouting was carried out to strengthen the right bank grout curtain extension and to elongate it further to the right. This work was carried out in 1987.

After Impounding of the reservoir, it seems that the sinkholes were developed after a considerable time downstream of the dam. Sinkholes were observed on the right bank about 900 m downstream of the toe of the embankment. Four sinkholes have appeared in a linear arrangement (Fig.1): SD2, SD2S, SD3-2 and SD4 (the largest). The sinkholes are not circular and have a characteristic elongated shape with a maximum dimension of up to 70 m and about half that in the normal direction. The longer dimension is aligned in the general line of the

sinkhole alignment which is approximately parallel to the dam axis in a northwest-southeast alignment.

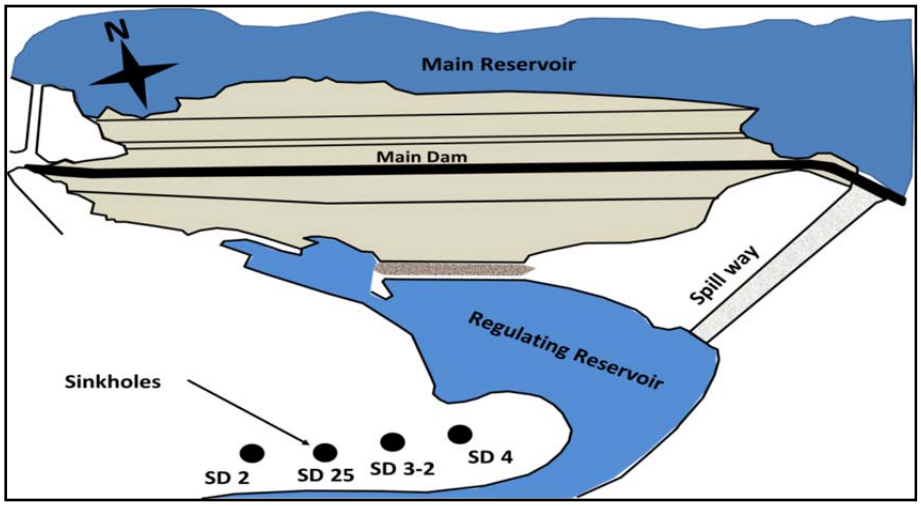

Figure 1: Sinkholes Downstream Mosul Dam –Right bank.

The sinkholes seem to be developed and appeared in 1992 and settled at a consistent rate until about 1998, when reduction in the settlement rate was observed to around half or less of the initial rate. The maximum measured settlement was about 5 m, and generally it was between 3 m and 3.5 m. In areas where the sinkholes are located beneath the concrete pavement, the deformed concrete may be bridging over settled ground at a lower level (Fig. 2). The average annual settlement was about 0.25 m per year, although for SD4 the average annual settlement was 0.38 m per year.

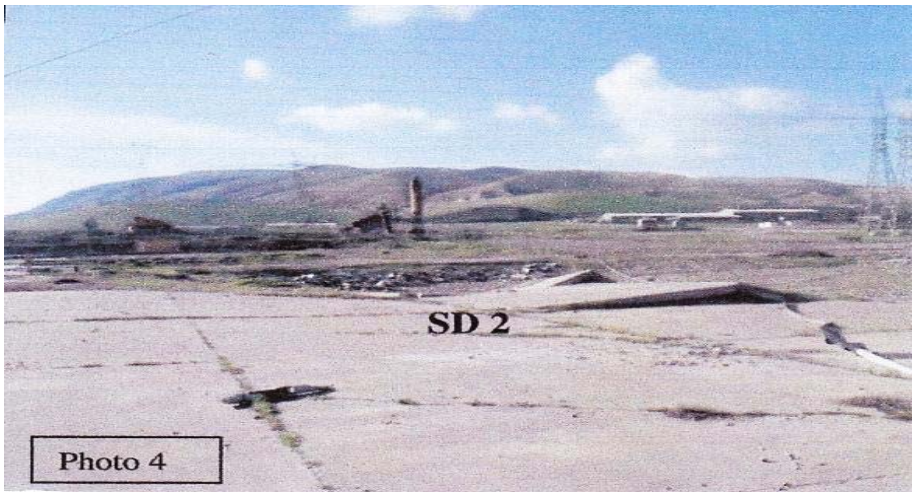

Figure 2: Sinkholes located beneath the concrete pavement.

The surface expression of the sinkholes suggests that they are caused by underground collapse. Concentric tension cracks appear to have developed around the central void as the sinkholes have developed progressively.

Sinkhole SD4 was excavated and left open for one year, allowing the site's staff to observe and measure water levels in it. It was found that the water level in the sinkhole does not respond to the changes in the water level of the main reservoir, however, it fluctuated with changes in the tail water level. Three factors tend to dominate when sinkhole activity is initiated or increased:

1. Increasing the infiltration rate of water at the ground surface.
2. Decreasing the groundwater piezometric level in the rock significantly below the soil-rock interface so as to provide an easy exit for the downward percolating water.
3. Repeated fluctuations of the groundwater below the soil-rock interface that alternately saturates and drains the soil.

The appearance of the downstream sinkholes is most likely related to fluctuations in the tail water level of the main dam during operation of the dam and the downstream regulating reservoir. It has been suggested by the site engineers that the sinkholes may be connected to the Darmellah aquifer on the right bank of the reservoir because before the impounding of the reservoir, substantial flows on the order of 360 l/s were encountered from a gypsum layer found during excavation of the tailrace tunnel for the pumping storage scheme to the west (further right) of the known sinkholes (Fig. 3); the water had a high sulphate content (which was different from the reservoir water).

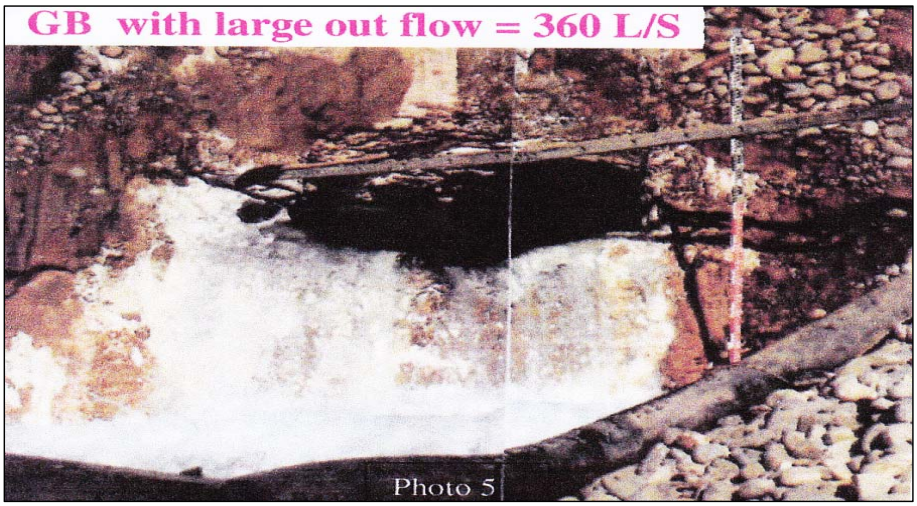

Figure 3: Flows with order of 360 l/s were encountered from a brecciated gypsum layer found during excavation of the tailrace tunnel for the pumping storage scheme to the west.

It also appears that the karst features within the bedrock may be better developed on the right side of the valley. Further evidence for connection of the sinkholes to the Darmellah aquifer is suggested by the appearance of a spring on the right side of the river channel. This spring, referred to as the right side spring (RSS) (Fig. 4), is in line with the sinkholes and appeared after erosion of the right bank during operation of the main spillway. The erosion has removed the alluvial deposits of unconsolidated silts and gravels; about 4 m thick overlying cemented conglomerate. When the conglomerate was initially exposed a spring appeared issuing from a fissure in the rock. The spring water had high sulphate content, and a flow estimated at 360 l/s, similar to the spring that issued from the gypsum bed in the tailrace tunnel. The water chemistry

was very different from the reservoir water and the existing springs on the left bank. The spring is reported to be generally not visible as it is below the water level of the regulating reservoir. For its approximate location refer to the arrow in figure 4 below.

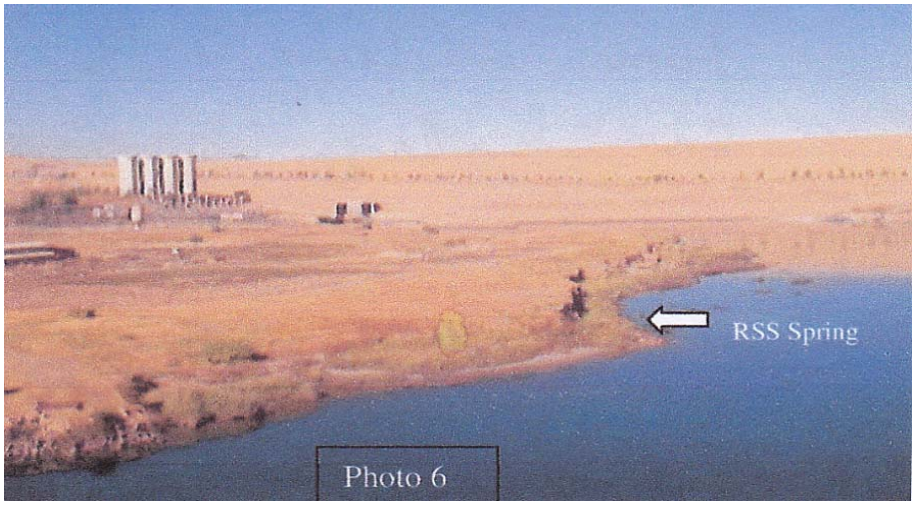

Figure 4: spring on the right side of the river channel.

Some 100 m upstream of the dam right abutment and on the slope above the end of the launching ramp (Fig. 5), the ground surface shows clear fracture. This fracture is caused most probably due to a subsurface karstification and cavern development. However, a slope failure may show same fracture, but then the fracture should be in crescent shape, not a straight one, as it is the case, as a result of fluctuating reservoir levels and residual high pore pressures in the slope. The concrete foundation for the steel fencing is also buckled and deformed. The only performed monitoring in this area was for vertical settlement; no measurements were performed that would indicate lateral movement of the existing three survey markers. The maximum cumulative settlement recorded was 2.46 m. Dye tracer tests did not establish a connection between the upstream depression and the downstream sinkholes.

Further upstream, on the next promontory on the right side of the reservoir a large conduit (small cave) was found in March 2002 within a gypsum/anhydrite layer, which crops out in the reservoir; it is unclear whether it is the GB1 or GB2 horizon. The maximum height of the cave is 1.3 m and it has clay infill on the floor (Fig. 6). The elevation of the cave floor is approximately 315m compared to a normal top water level of the reservoir at elevation 330 m. Both the cave inlet and outlet are within the reservoir pool and are normally submerged from January each year for about 8 months of the year.

Figure 5: Open fissure resulting from slab slide upstream right abutment of the dam.

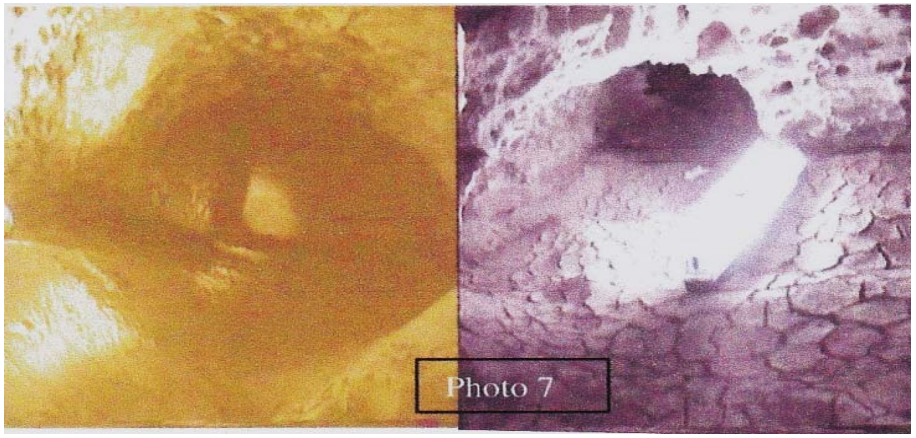

Figure 6: Large conduit (small cave) was found in March 2002 within a gypsum/anhydrite layer on the right side of the reservoir.

Al-Ansari et al. (2013) and Issa et al. (2013) conducted a bathymetric survey during (2011) and concluded that *"After 25 years of the dam operation following; the thalweg bed slope of the River Tigris had changed from 0.65 m.km^{-1} before dam construction to 0.71 m.km^{-1}"* during the 2011 survey. The sedimentation rate in the upper section of the reservoir where the River Tigris enters the reservoir was the largest and gradually decreased toward the Mosul Dam site. The maximum deposition thickness was 17.6 m in the upper zone of the reservoir. Furthermore, there are many areas like the middle and lower parts of the reservoir that are believed to have suffered from dissolution of gypsum/anhydrite beds forming sinkholes that might reach about 20 m in diameter and 9.6 m in depth. The conducted survey also confirms the presence of sinkholes and other karst forms within the reservoir, which are active, continuously increasing in size, and exerting hazards to the Mosul Dam.

The sinkholes on the right abutment have developed progressively with initial manifestation by surface cracking and ground settlement. By contrast a sinkhole on the left bank of the reservoir in the local tourist village appeared without any indication and developed rapidly with depth of 15 m and a 15m diameter depression overnight in February 2003 (Fig. 7). The sinkhole was dry, located on a slope on the northern side of a valley feature. The deposits at the surface

standing vertically appear to be superficial silty soils, with perhaps bedded marls below. Although the initial sinkhole was filled with 1200 m^3 of loose sandy gravel material and fenced off, the settlement was continued. A further 3000 m^3 of material was required to fill the settled surface of the sinkhole in two separate filling operations in May 2003 and October 2004. By March 2005 there has been a small settlement of the ground surface for about 0.5 m.

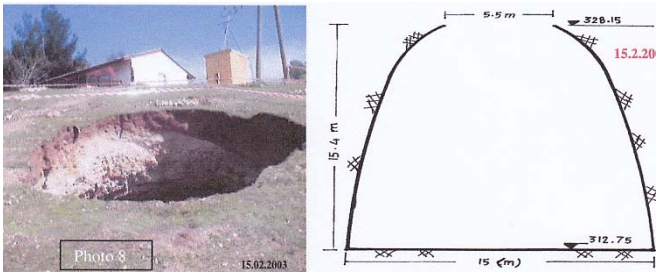

Figure 7: Sinkhole at left bank of the reservoir.

The surface expression of the collapse with the absence of concentric tension cracks and sudden appearance suggests that the collapse resembles a crown hole. It was reported that during the drilling of a piezometer hole near the sinkhole a rod drop had occurred. There was no water encountered in the hole so the piezometer was not installed and the hole was grouted, consuming 250 tons of grout. The suggestion has been made that the sudden appearance of the sinkhole is related to seepage water passing through the left abutment of the reservoir and the sinkhole lies close to the end of the left extension of the original grout curtain. Though possible, an increase in infiltration of surface water, perhaps from heavy rain, could cause a collapse into an existing void by destabilizing the ground. Since the appearance of this sinkhole the grout curtain has been extended eastwards but the grout consuming was not high.

Voids at the surface as evidences for the presence of sinkholes and ground settlement are of substantial size, and suggest large voids have been present below the surface to allow the ground surface to settle. An initial impression of these sinkholes is that they are not directly related to the deep foundation issues beneath the dam. There have been no reports of large rod drops indicative of big voids. Clearly some of the grout consumption reported suggests that there must be some Interconnection of conduits or voids associated principally with gypsum karst or that voids are being developed either by erosion or by solution.

A common feature of the sinkholes is that they are found to be associated with natural valley features. It appears that the known sinkholes are geologically controlled rather than related to seepage. Ground collapse may have been initiated by changes in groundwater levels due to impounding of water in the regulating reservoir for those in the valley downstream of the main dam. The alignment of the sinkholes downstream of the dam strongly suggests a geological (structural) control and relatively near surface phenomenon. However, it has not been possible to verify this either by carrying out detailed geological field studies at the site or by obtaining high quality aerial photographs or Landsat images. The recent collapse in the tourist village on the left bank has the characteristics of a crown hole formed by sudden collapse into a pre-existing void. The trigger for the collapse is not clear; it might have been purely time dependent, or due to

some external factor such as rainfall, or changed ground water regime due to reservoir impounding.

The locations of all sinkhole suggest a linear arrangement crossing the valley downstream of the dam (Fig. 8), although it has yet to be determined what that feature might be. The site personnel have noted initial indications of what may be possible indications for development of sinkhole in future; further east along this line. In terms of the impact of sinkholes on the dam, we are not aware of any current technology that could be used to detect the presence or the future development of voids or potential sinkholes beneath the dam embankment structure, either from the surface or from within boreholes. Vertical extensometers below the grouting gallery might give some indications for settlements of the strata that might occur below the gallery; however, the best way of detecting changes in the foundations remains "the use of piezometers". The use of multi-point piezometers would improve the monitoring of groundwater changes in the foundations and permit sampling of the groundwater for groundwater chemistry and flow studies. In the downstream area and village it is conceivable that a seismic reflection and/ or microgravity study (see Hijab and Al-Jabbar, 2006 and Hijab, et al.. 2007). A ray may detect large caverns or voids in the subsurface that could develop into sinkholes.

Figure 8: Mosul Dam, the dashed line shows approximate alignment of the sinkholes.

3. Seepage and Dissolution of Gypsum

In winter 1986, the reservoir level started to rise and impounding for the first time began to take a rapid tempo. At the same time percolation through the dam foundation manifested itself by the occurrence of six major springs on the left bank at different elevations along the 1.5km long stretch. Seepage was also occurring under the dam in the deep river section which was submerged by tail water. Safety concerns required adequate surveillance of seepage and checking of the dissolution of gypsum from the foundation.

Measurements of the discharge of the surface springs were systematically observed and measured. Permanent discharge measurement weirs with water level gauges were built at all important surface spring areas. The seepage quantity in the river channel was measured by converting coffer dam no.6 which was used during river diversion in to a measuring pond with

an overflow weir built in to it. Water samples were also taken at two weeks interval and analyzed for their mineral content to establish the quantities of gypsum being leached from the foundations. Results of seepage surveillance from February10, 1986 to August16, 1986 showed that the recorded seepage through the Main dam foundations and the left bank increased from 500 l/sec to 1400 l/sec in which water head had increased from 49 m to 65 m. Springs (Sa) and (Sd) in the left bank alone had increased from 150 l/s to 900 l/s (Fig.9).The submerged springs at the river channel showed an increased transmissibility by 40% during the same period (from 630 cubic meter per day to 880 cubic meter per day) (Guzina et al., 1991).

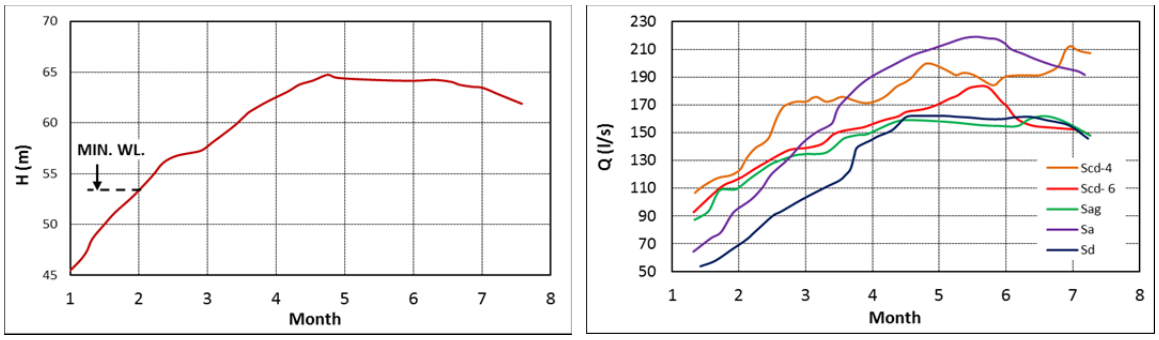

Figure 9: Left: spring discharge and Right: Reservoir water level for the period February-August, 1986.

Dissolution of minerals from Dam foundations was established through the difference in the content of dissolved minerals in seepage water in relation to the water in the main reservoir (amounting to 250 mg/l as the average). The total mineral content of the seepage's water by individual spring zones is presented in (C) and (D) of Fig.10). Analysis of these results yielded that 13 000 tons of minerals were leached from the dam foundations at this period and that 70% of this quantity originates from the submerged springs. The dissolution intensity ranged from 42 to 80 t /day.

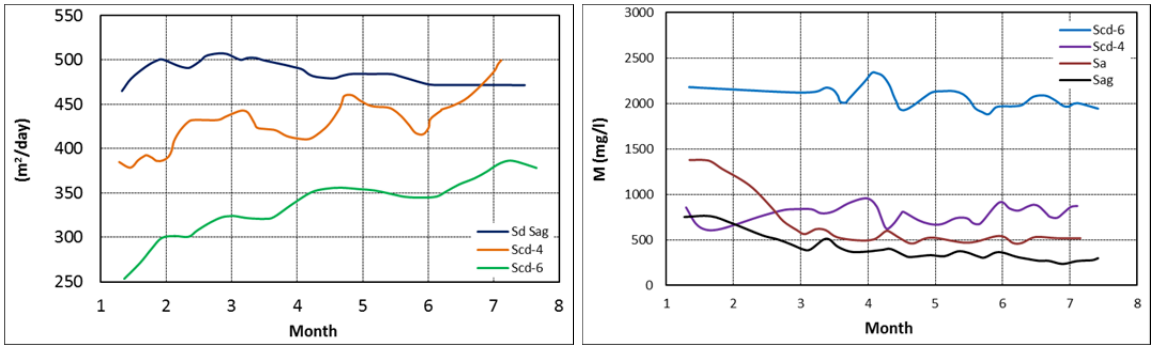

Figure 10: Left: Spring transmissibility and Right: soluble salt concentration for the year 1986.

These chemical analysis results pointed out to significant dissolution of gypsum and anhydrite followed by noticeable increase in the permeability and leakages through foundation. Analysis of mineral content of seepage water pointed out to the Gypsum and anhydrite and other minerals were being washed out from joints and fractures crossing insoluble rocks and it was expected in view of the repeated maintenance grouting that all gypsum present in joints would be washed away from the zone of grout curtain. Bearing in mind that some of the rocks in the foundation

are erodible then this dissolution of gypsum which is present along fractures and faults could initiate further erosion along those discontinuities also across the zone of the deep curtain. Direct dissolution of gypsum and anhydrite layers, which could occur at the contact of these layers with permeable rocks (such as limestone) will result in the formation of caverns extending upstream and could by the progress of time jeopardize seriously vital project structures. On the other hand, the dissolution may occur along fractures in layer or bank of gypsum or anhydrite within impervious layers leading to a dangerous situation especially when the layers are in contact with the reservoir and tail water. In such case karstification processes would begin on the upstream side and move progressively in the downstream direction.

In view of the above, much concern was raised over the dam safety and the situation was followed closely by the International Board of Experts for Mosul Dam (IBOE). During the following two years, many remedial measures were recommended and put in to action which included additional grouting along the dam axis in the left embankment by deepening and strengthening the grout curtain, elongation of the curtain extension beyond the left end of the dam and constructing a new grout curtain alongside the left side of the spillway bucket to cut off the seepage flow from the left passing under its foundation which is in contact with gypsum-breccia layer (GB3).

The deep grout curtain also received its share of attention and works were intensified in an attempt to improve its quality which remained an open question up to now. Other minor works were also performed such as collecting the flow of the springs in three measuring points, (Fig. 11). In addition to the coffer dam number 6 measuring pond, and covering seeps and wet areas by filter material. From figure 11, it can be noticed that:

a) Point (1), at the right side of the spillway at the end of a collection channel which collects seepage water from under the spillway that seems to originate from left side.
b) Point (2), at left side of the spillway collects springs between seepage from under the dam further left and around the left side.
c) Access Gallery end point collects seepages from under main dam from the river section to the spillway.
d) Coffer dam no.6 measuring pond measuring seepage under the deep section of dam.

Measurements of seepage quantities and mineral content of water have been conducted over all these years .These measurements show stable conditions and no adverse effects, but they also indicate strong correlation with the reservoir water levels. Dissolution under the dam and its close proximity also continues at smaller rates due to the nature of the foundation (National Council of Iraqi Dams, 2002).

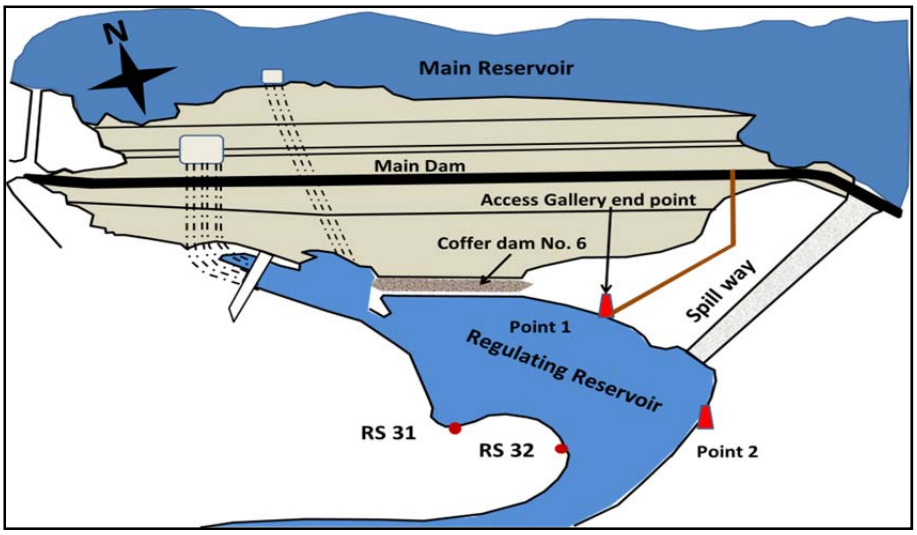

Figure 11: Seepage water Collection Points at Left Bank and cofferdam no.6 Measuring pond (Public Authority for Dams and Reservoirs, 1995).

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CHAPTER 5: FOUNDATION TREATMENT DURING CONSTRUCTION

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1. Introduction

In solving the seepage problems expected in any new dam, foundation treatment plays a great role in preserving its safety and integrity. In Mosul Dam this takes even greater and more important function. This is attributed to the complexity of the geology and the presence of soluble gypsum and anhydrite in the form of primary gypsum/anhydrite in thick layers and also as secondary gypsum/anhydrite present in joints or mixed with fine clays in whole gypsum breccia layers. In addition to the existence of weathered layers of limestone which are characterized by a lot of cracks, fissures, joints and cavities which make them very pervious and erodible.

Grouting in gypsiferous formations is a very tricky operation. Because when such operation begins to seal some seepage paths, this will result in an increase of hydraulic gradient locally in others. James and Kirkpatrick (1980) explained that water passing over gypsum becomes chemically saturated within a flow path and in this zone of saturation no further dissolution occurs. As flow continuous, the zone moves downstream and eventually passes from the exit. At this stage, dissolution rates accelerate again sharply. Results of studies of Morrison-Knudsen Engineers Inc. (1989) also confirmed James and Kirkpatrick comments regarding sensitivity of gypsum solubility to hydraulic gradient and flow. Their report indicates that for seepage velocities of 10^{-4} cm/sec in a 2 cm wide gypsum vein should dissolve at a rate of few centimeters per year from an advancing front. If the velocities were about 10^{-2} cm/sec the gypsum could dissolve at a rate of 9 meter per year. Dissolution occurs until seepage water reaches a calcium saturation of 2000 ppm. Hence the dissolution zone moves downstream as greater quantities of unsaturated water attack a gypsum vein.

From Soviet experience gained by soviet engineers from the design and construction of dams in eastern Siberia and central Asia on gypsiferous foundations it is permissible to build such dams provided that these gypsiferous rock structures are with permeability of not more than 0.1 m/day (4 $x10^{-4}$ cm/sec) (Nedrigs and Yanova,1986). The same authors cited also the case of the Kama dam on the river Kama in which the upper part of its foundation to a depth of 50 m is composed of hard and soft rocks represented by sandstones, argillites, limestone, dolomites, and marls, and the lower part by sulfate complex in the form of beds of compact gypsum and anhydrite with a thickness up to 120 m. The dam was successfully built with protection measures against seepage and piping consisting of an upstream clay blanket of 100 meter length, and a deep grout curtain connected to the blanket at the upstream and a drainage system to localize seepage flow. This arrangement provided reliable operation of the structure for 30 years after which it became necessary to conduct works on strengthening and maintaining the grout curtain. From all these it seems it is most difficult to seal a cracked or fissured gypsum formation permanently, especially in the presence of other formations which are also jointed, cracked and are highly conductive to flow as in Mosul Dam foundations and in the view of the very high heads created by the reservoir.

Nevertheless, the designers of the dam considered that grouting should be used as the anti-seepage element for the deep cutoff under the dam, while construction of positive cutoff in the form of concert diaphragm could have been used instead. Hydrofraise machines for the

construction of such diaphragm to a depth of 100-120 m from the river bed level were at use at that time in the world.

2. Grouting Works Details

The treatment of the Mosul Dam foundations consists of two main elements:

a) Blanket grouting.
b) Deep grout curtain.

2.1 Blanket Grouting: This was performed under the core of the main dam. It was intended to close the openings originally existing in the foundation rocks, creating thereby a more homogenous upper part of the foundation with respect to the permeability and compressibility. The blanket grouting should also create a bulk head at the top of the grout curtain and should elongate the seepage lines, in addition to closing any preferential path of seepage at the contact of the core with the foundations. Permeability tests were conducted using Lugeon test in exploratory holes before starting grouting operation to find out later the efficiency of completed work. High permeability zones were found with average values of 28.7 Lu on the right bank from Sec. 87 to Sec. 113 and on left bank an average value of 54.5 Lu from Sec. 64 to Sec.86 (Fig. 1).

The Lugeon test, sometimes called also Packer test, is an in-situ testing method widely used to estimate the average hydraulic conductivity of mass rock and the results provide information about horizontal conductivity of the rock mass including the rock matrix and the discontinuities. The test is named after Lugeon (1933), a Swiss geologist who first formulated the test. For details and description of procedure consult Lancaster-Jones (1975), US Department of the Interior- Bureau of Reclamation (2001), Camilo Quiñones-Rozo (2013) or geotechdata.info (2014).

The details of the performed blanket grouting are given in table (1). Final evaluation of the quality of the completed grouting after finishing the basic pattern (P, S, T) holes and even after performing quaternary and quinary holes showed that some sections of the blanket did not meet the acceptance criteria. These were mainly in Sec. 115 and Sec. 110 on the right side with residual permeability of 24.1 Lu and 20.8 Lu respectively in chalky limestone beds and in Sec. 65 and Sec. 69 on left bank with permeabilities of 11.6 Lu and 11.7 Lu in the transition zone of gypsum/breccias layers GB3 and GB2 (see Fig. 1) and very likely in limestone bed intercalated in clayey series. These results show the possibility of some deterioration of the blanket at these locations in the future.

2.2 Deep Grout Curtain: It is meant to create a barrier against seepage flow in the foundations under the dam and reduce the permeability of the grouted zones to the possible minimum values. This is intended to hinder or even stop the dissolution of gypsum and anhydrite layers in primary form and secondary gypsum in joints, fissures or cavities in other erodible layers. It was also meant to plug joints, cracks, fissures, and fill the present cavities in the limestone in the F-beds and in the chalky series reducing the general seepage flow in the foundation and reduce any erosion process in these rocks.

It is very clear from the extensive geological drilling performed during the investigation phase that the geology along the dam axis has been subjected to much tectonic movements and disturbances, in addition to effects of weathering. All these factors required that the grout curtain

should adapt to the variable geology along the axis and also to the variation of water head along this axis. Based on this the curtain is divided in to four parts, namely; 1) the extension of left bank, 2)the Saddle dam and fuse plug ,3)the deep grout curtain under the main dam and 4) the extension at the right bank. All these parts were carried out from the ground surface except the part under the main dam (Valley floor and abutments) which was performed from the concrete grouting gallery and its extension tunnel at the left abutment, while grouting under the spillway head work was performed from the gallery in the lower part of this structure. Table 2, describes the main features of these parts including the extent of each, the targeted formations and the specific goal intended to be achieved.

The construction of the curtain under the main dam was done from a concrete gallery extending from the right bank to the left bank. It was built in open cut at the bottom of the cut off trench of the dam and it is provided with an access tunnel leading to the left bank ground surface and another access from the right bank. The gallery itself continues in the left direction as a grouting tunnel which leads to the gallery under the spillway head work. Pairs of peizometers u/s and d/s of the curtain were installed in the gallery to monitor the performance of the grout curtain during operation and they proved to be very useful in the stage of maintenance grouting to locate deteriorating areas.

The depth of the curtain along the dam axis had to be decided by drilling a deep exploratory hole in each section with 100% core recovery and carry out a water pressure test to check the permeability of each of the layers encountered and correlating the results with the type of rock and the quality of the retrieved rock cores. Reduction of permeability to safe limit within the treated depths should be the end result of the grouting operations.
The accepted limits of residual permeability were set as in table 3.

Table 1: The details of the performed blanket grouting.

Location	Arrangement Details	Type of Grout Mix	Acceptance Criteria
Under the Main dam core from Section 64 on the left abutment to Section 113 on the right abutment (length of section = 36m.)	It consists of: - 10 rows of holes U/S of dam C/L. - 10 rows of holes D/S of dam C /L.	Cement-based mixes ranging from mix A to mix D with bentonite as additive	Using water pressure test in drilled holes in the finished work
Treatment carried out from foundation surface (no counter weight) or in some cases from intermediate stage of Excavation (with limited counter weight)	Depth of holes : - 25 meters for internal row holes. - 10 meters for external row holes.	Mix A C/W=0.25 C/B=25 (thin mix) Mix D C/W=1.0 C/B=25 (thick mix)	- 90% of all tested stages should give < 10 Lugeon - No stage should be >30
	Spacing between rows is 2 meter. Spacing between holes in the same row is 3 meter for primary holes with split spacing for secondary tertiary and possible quaternary and quinary holes if required.	C= cement weight W= water weight B= bentonite weight	Based on quantity of grout takes and engineering-geological judgment. Upon completion of the basic pattern, the decision was made whether to drill additional quaternary holes and quinary holes

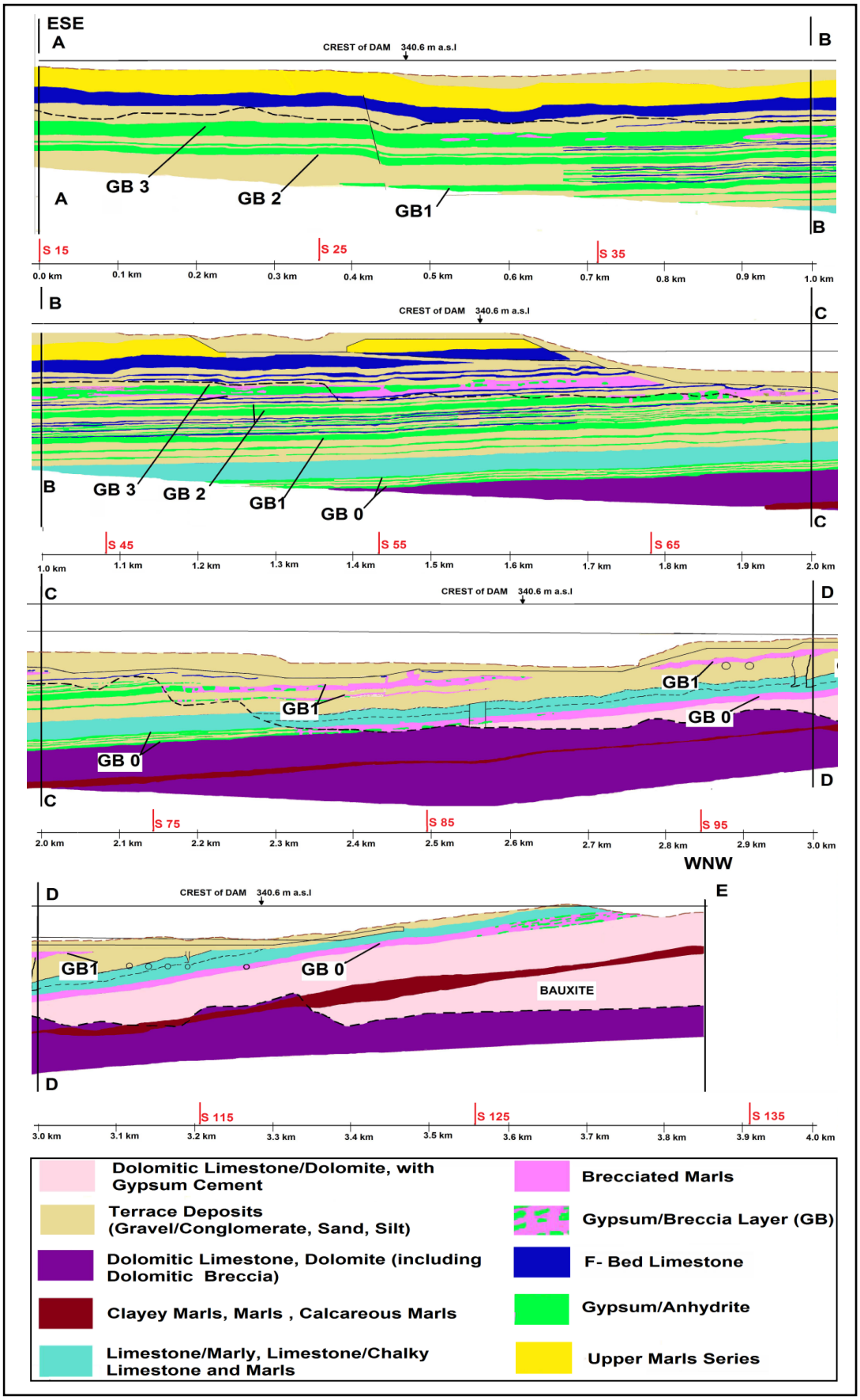

Figure 1: Geologic cross section along the axis of the dam.

Table 2: Parts of the Deep grout Curtain and their Characteristics.

Part of curtain	No. of Rows	Targeted Formations	Function
Extension of Left Bank From Sec.215 to Sec.151 Total length 1560m (Sec. Length 24m)	1 Row Done from ground surface	The foundation here up to 20-30m depth is highly pervious especially the F-bed limestone	To limit seepage flow through abutment from Gebel Taira anticline to the end of saddle dam
Saddle Dam and fuse plug From Sec.16 to Sec. 47 Total length 1152m (Sec. Length 36m)	2 Rows One row was originally designed but a second row was added in 1986 after appearance of springs at spillway bucket area.*	Extends through sand silt gravel deposits then in the upper marl series and finally through the fairly thick pervious F-bed limestone bed	To limit seepage to the area between the end of main dam and in the spillway area and below the fuse plug saddle dam
Main Dam From Sec. 48 to Sec. 114 Total length 2379m (Sec. length 36m)	3 Rows of vertical holes and; 2 Rows of inclined holes done from the sides of the grouting gallery. Length of these holes 25 m.** Drilling and grouting works are carried out from a concrete grouting gallery and grouting tunnel under left abutment.	The vertical holes down to 80-100 meters to penetrate all beds especially the pervious and soluble beds to reach the karsts level.	The vertical curtain to minimize seepage flow to safe limits to stop dissolution and erosion processes in all layers above karst level. The inclined holes to create a tight contact zone between the blanket and top of the curtain
Extension Right Bank From Sec.123 to Sec.139 Total Length 408m (Sec. Length 24m)	2 Rows from Sec.123 to Sec.132, and 1 Row from Sec.132 to Sec. 139		Limits seepage flow around right abutment. But It did not extend enough neither laterally nor in depth to reach low pervious rock. The length of holes reached more than 100 m.

*These springs appeared in February 1986 at the filling of the reservoir for the first time and resulted from the seepage under the dam and passing under the spillway foundation and threatened its stability. Therefore it was necessary to strengthen the curtain here.
** The inclined holes were meant to improve contact between the blanket grouting and the top of the grout curtain.

Table 3: The accepted limits of residual permeability.

Extension Left	Saddle Dam and Fuse Plug	Main Dam	Extension Right
No specific Lugeon value required	90% of all stages < 5 Lu 100% of all stages < 10 Lu	Upper 30 m 95% of all stages < 2Lu 100% of all stages < 5Lu	No specific Lugeon value required.
Acceptability is judged from the quantities of grout take and types of foundation rocks		Below 30 m 95% of all stages < 5 Lu 100% of all stages < 10 Lu	Acceptability is judged from the quantities of grout take and types of foundation rocks

2.2.1. Main Dam Curtain. Full picture of permeability variation under the dam and along its' axis was made by carrying out the usual water pressure tests in the drilled exploratory holes as a first step. This variation is shown in table 4 at the various formations from lower to upper units.

Table 4: Variation of Permeability in Various formations under the main dam.

High Permeability	Moderate to low Permeability	Low to Nil Permeability
In Dolomitc limestone above the well-defined karst level*	In formations such as clayey series, GB layers as fossil karst and upper marl series above karst level	All formations below Karst level
In GB0 on the right bank		
In chalky series in valley floor and right bank(Sec.78 to Sec.114) above karst level		
In GB3 layer and In transition zones above karst level		
In Isolated Limestone intercalated in clayey series above karst level		
In F-bed limestone in left bank above karst level		

*for karst level definition refer to (Swiss Consultants Consortium, 1989).

After completing the basic grouting pattern with (P, S, T) holes the need arose for adding quaternary and quinary holes. Residual permeability values in many of these areas remained above the acceptance criteria .These locations were called "Windows " and were located in the following parts of the curtain as follows:-

-Large areas in the Jeribe limestone from Sec.89 to Sec. 109 (Valley floor and right bank).
-Large areas in chalky series from Sec. 79 to Sec. 92, and Sec. 97, 102, 104, 106 and Sec.108.
-Large areas above the grouting tunnel between Sec. 57 and Sec. 61 as well as limited areas in Sec. 55 and fairly large area below the spillway head structure, Sec. 49 to beginning of Sec. 54.

Since the end of grouting activates several of these zones were treated again by additional holes (Maintenance grouting) and this work continues even up till today. The grouting mixes used were basically (Cement Bentonite mixes) but bentonite gel mix and silica gel mix were also used in quaternary and additional quinary holes when the basic pattern continued to show high residual permeability.

At later stage (November 1987) Massive mix was used when some of the completed areas were showing deterioration and high grout takes were required and in a very short time (Rodio-Keller and Mosul, 1987). This mix consisted of two weight of sand and one weight of cement plus 4% bentonite. The need to deliver large volumes of this mix in relatively short time required drilling one service hole from the top of the dam to the gallery and encasing it with 15cm steel pipe. Grout mix was delivered to the intake by trucks and pumped through the pipe and it was possible to grout in more than one location in the gallery at the same time. This method proved to be very useful in emergency situations and also during the maintenance work up to now as indicated by the Mosul Dam International Board of Experts, (IBOE, 1988a). Two more such delivery pipes were installed later on at other sections.

2.2.2. Saddle Dam and Fuse Plug Curtain. Exploratory holes with 100% core recovery and water pressure tests were also completed to define the depth of curtain. High permeability

zones were located in the F-bed limestone and in the under laying anhydrite / gypsum breccias GB3. After completion of the two row curtain, limited zones had residual permeability higher than the accepted design criteria. These were located between Sec.31 and Sec. 35 in addition to Sec.38.Two areas where in F- beds while three other areas in GB3. Cement base grouts were used in the basic (P, S, T) pattern and bentonite gel mix was used in the quaternary and quinary holes which were drilled after recording high takes in the basic (P ,S ,T) pattern.

2.2.3. Left extension Curtain. The one row curtain at this part extended over a distance of 1500m. It was meant to treat highly pervious F-bed limestone formation. Only 12 exploratory holes were drilled to depths between 59m to 120m. Full core recovery was performed with water pressure tests to define approximately the limit between pervious and impervious rocks. This indicated the presence of high permeability areas in the F- bed limestone .On the contrary the GB3 and the upper marl series showed very low permeability.
After completion of the grouting holes using cement base grout only no check holes were performed. But quantity of grout consumed was used as indicators of the efficiency of treatment. High consumption were recorded in Primary and Secondary holes and in many Tertiary holes between Sec. 201 to Sec. 198, moderate takes to low takes from Sec. 197 to Sec. 189. In sections 182,180 and 178 grout takes were so high that quaternary holes and quinary holes were also performed.

2.2.4. Right Extension. This extension extends from end of the dam at right bank Sec.123 for a length of 408 to Sec.139. It consists of two row curtain from Sec. 123 to Sec. 133 and one row curtain from Sec. 134 to Sec. 139.

Exploratory drill holes were performed with full core recovery and water pressure tests to define the line between pervious and impervious rocks. The depth of the upstream row in the two row curtain varied from 135m to 125m, while the second row depth varied from 90 m to 60 m. The direction of the extension follows slightly towards upstream. The length and direction were selected as a result of hydro geological study based on a mathematical model (Swiss Consultants Consortium, 1984) and also by using the water pressure tests. Parent rock permeability results had showed very high permeability values in the dolomitic limestone above and below the karts level which remains very deep and runs in a perpendicular direction to the extension. No check holes were drilled after the completion of this extension but examination of the grout consumed quantities showed average value range between 295 kg/m at Sec. 123 and 944 kg/m at Sec. 127.

3. Evaluation of the Foundation Treatment at End of Construction

The deep grout curtain construction was completed on February 6th 1988. But maintenance and repair works which were started in the end of 1986 continues up to now. Difficulties were encountered in reaching the approved acceptance criteria since 1984 but these works indicated since 1986 through 1988 the deterioration of the curtain in many locations (IBOE, 1988b). At the end of 1986, a campaign of “Cement Groutability Tests” was performed with the view of closing the Windows. The high consumed grout quantities experienced led to the adaption of the

"Massive Grouting Method" in addition to the normal grouting, for the maintenance works, (IBOE, 1988c, d).

The Swiss Consultants evaluation of the works carried out at the end of 1988 without considering the maintenance works gave the following conclusions (Swiss consultants, 1989):

1. The results of grouting treatment of the dam foundation showed obvious correlations with the prevailing geology and a well-defined limit between pervious and impervious rocks (karst Level) was established. This limit follows the morphology of the valley floor and abutments.

2. After completion of the grouting works, lenses of high residual permeability (i.e. above the accepted design criteria) still existed within the Main Dam foundation in the Jeribe limestone and Chalky series.

3. In the calcareous formations, particularly in the chalky series, the still pervious matrix left is very fine grained; its permeability cannot be reduced further by grouting using cement grout mixes, but only by chemical treatment. On the other hand there are no soluble rocks within these zones, which could lead with time to a permeability increase. For these reasons a general additional treatment by chemical grouting was not justified in the chalky series and the Jeribe limestone which are at considerable depth, and therefore such additional grouting was considered not necessary.

4. High residual permeability cannot be tolerated in anhydrite/gypsum beds, especially in zones where the dissolution process had started, but fairly high percentages of gypsum was still contained. Such transition zones must be permanently controlled. Sign of seepages through the grouted rock shall be stopped without delay by additional local treatment.

From the forgoing it is established that an extensive maintenance program is necessary to control the seepage process within the grouted zone to stop dissolution of gypsum and protect the safety of the dam. The total amount of grout injected since 1986 is about 95657 tons.

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CHAPTER 6: MAINTENANCE GROUTING

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1. Introduction

The Mosul Dam grouting works were completed on February 6th, 1988. Maintenance grouting works; however, has been going on since September 1986, at which time impounding of the reservoir was started and water level was rising steadily. Figure 1 shows the quarantines of grouting takes injected from October 1986 up to October 2014. Efforts at that time were going on to close the already unsealed so called "Windows", where design criteria for acceptable residual permeability could not be reached. Meanwhile, areas of high grout takes appeared in other locations. The general accepted cause for this was the dissolution of the gypsum and/or anhydrite beds resulting in the deterioration of the curtain. So, it was clear that new methods had to be found to cope with the new situation. High hydraulic heads were steadily building up resulting in increased seepage flows through the curtain. New grout mixes were tested and even methods of delivering and injecting large grout quantities were developed. Maintenance grouting works continued from September 1986 up to now. Where the quantity of grout used from 1986 to 2015 reached 95657.43 tons. Techniques of Enlarged (massive) grouting as it is called now are used in addition to normal grouting that was already in use.

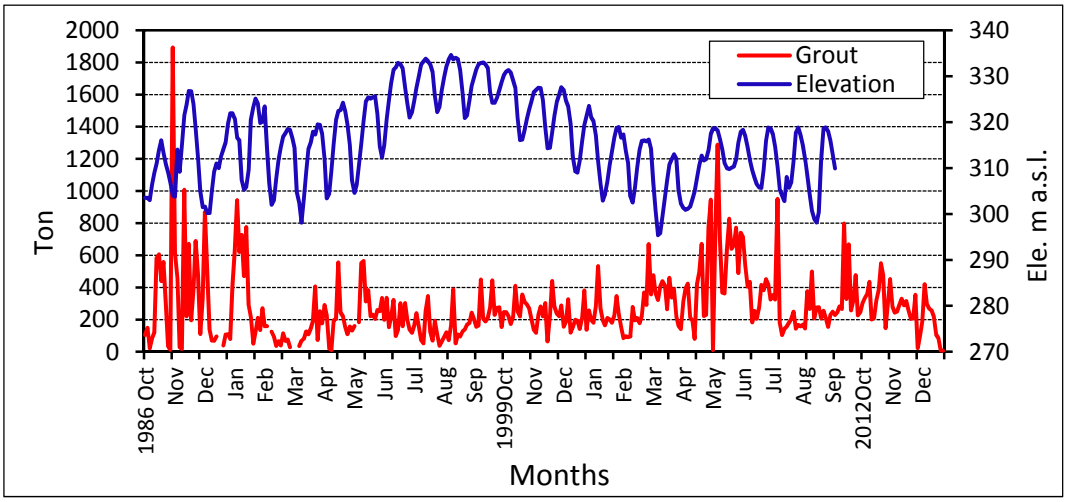

Figure 1: Monthly grouting and water level at Mosul Dam.

2. Groutability Tests

In this direction, much consideration and testing were performed during 1987 and 1988 to reach a satisfactory method for treating large takes areas to ensure dam safety. This was called the" Groutability Tests Program". Chemical grouts such as gelling mixes and silicate mixes were overruled over fears of toxicity and inadequacy in case of existing large cavities; in addition of high cost due to the large required quantities. Concrete use was investigated also and the concrete mix designated as type CT13 was tried. This mix has the following proportions;

Cement weight = 500 kg.

Sand (0-3 mm) weight = 694 kg.

Gravel (3-8 mm) weight = 1046 kg.

Water volume = I L

Total dry weight is 2240 $kg.m^{-3}$

The use of this mix was abandoned due to technical difficulties and low productivity rate. Attention was turned then to use sandy mixes (SS) in addition to the cement mixes, which were already in current use. Sandy mixes were developed by adding certain weight of sand to the cement mix. And even pouring gravel after completion of grouting in large takes' zones. The performance of the additional grouting was done by drilling multiple patterns of bore holes (3rows) at 1.5 m of hole's spacing. The new rows were to be located between the middle row and downstream row of the original curtain. Depths of these new holes should be 20 m below the original curtain and further deepening was instructed, where large grout take was experienced in the last grouted stage.

Two alternative sequences of the grouting operations were experimented and results were evaluated as follows:-

i. To grout the three rows and pour gravel (3 - 8 mm) in all boreholes during injection of the grout mix to create a "Matrix".
ii. To grout the upstream and middle rows and pour gravel in the third row while injecting in those holes to create a "barrier".

In evaluating the results of the gravel addition, it was concluded that it is not effective and very difficult to pour, and that many high take zones were still not sealed after completing the grouting operation, so a decision was taken to discontinue its addition. Concerns were voiced as whether gypsum/anhydrite dissolution was occurring at a rate higher than the filling by cement and sandy mixes and pouring of gravel. The final decision was a to try and inject large quantities of sandy and cement mixes in high take zones at a very high rate and by doing so the grout would fill the zone under consideration quickly and overcome the seepage flow and stop it. The industrial production and injection of the cement and sandy grout mixes was tried and it was called "Enlarged" or "Massive "grouting (Rodeo-Keller, 1988).

3. Massive Grouting

The proportions of the sandy mix (SS) were checked and after some trials the following mix design (SS) was thought to give best results:

Cement weight = 465 kg.
- Sand / Cement ratio ≈ 2:1, so sand weight is 930 kg.
- Water / Cement ratio = 1:1, so water is 415 L.
- Bentonite / Cement Ratio = 4%, so bentonite weight is 18.6 kg.

The total dry weight was 1413.6 $kg.m^{-3}$
The grading of used sand was 1 to 4 mm.

In order to reach very high capacity for delivering this grout, mixing of the bentonite slurry was performed first in the grout mixing plant and then loaded in concrete truck mixers. The adding of cement and sand was then performed in to the mixers in the concrete batching plant. The grout mix was then transported from the concrete batching plant to the crest of the dam at section 77 and pumped down to the grouting gallery through steel service pipe placed in a bore hole 109.5 m long. This hole was drilled through the core in to the gallery. The pipe was black steel pipe OD 12.7 mm. and ID 109 mm of flush joint type. At the bottom; inside the gallery an

energy dissipater reduced the velocity of the pour mix and distributed it into an agitator tank with a capacity of approximately 1 m^3 and to a grouting pump, which fed the mix into the injected hole by a high pressure flexible hose. This arrangement increased the delivery rate of grout 10 times more than the previous method and reached an average quantity of 20 m^3 per shift.

The maximum pumping distance inside the gallery was in the order of 180 m in each direction. This limitation was caused by friction encountered by the mortar during its flowing in the injection pipeline. As this arrangement proved its efficiency, a second bore hole was driven and a service pipe was installed at section 71. A report states that a third service pipe was also installed later in section 88 to cover the whole length of the problematic area, and a peak value of 400 m^3/day had been delivered in some instances (Public Authority for Dams and Reservoirs, 1995).

4. Detection of Problematic Zones

Piezometric observation in Mosul Dam grouting gallery proved to be an exceptionally valuable tool for checking of the conditions of the grout curtain and the detection of problematic areas where additional treatment was required. For any pair of piezometers, if the pressure difference decreases, then this indicates an increase in seepage flow through the curtain at that location and the formation of a new dissolution path or cavity. It may be said; therefore, that this loss of head is a good indicator in such practical cases. In an electric-analogy study carried out on the curtain of a high dam in India (Goel and Sharma, 1984) the following relationship was used for the evaluation of the deep grout curtain (Efficiency) efficacy:-

$$E\ \% = \frac{H-h}{H} \quad \text{.....(i)}$$

E is the Efficacy (Efficiency) %
H is the reading of the U/S piezometer
h is the reading of the D/S piezometer
H-h represent the loss of head, and when h → H then E → 0
In this equation, H and h are measured from the same datum.
In this direction a similar procedure was used in Mosul Dam (General Meeting on Iraqi Dams Safety, 2000) to establish areas of grout curtain deterioration and therefore, locate the dissolution zones. The efficiency index was determined as follows:

$$EFF\ \% = \frac{U/Sp-D/Sp}{R.L-T} \quad \text{.....(ii)}$$

U/Sp is the upstream piezometer reading
D/Sp is the downstream piezometer reading
R.L is the Reservoir level
T.L is the tailrace water level

After the operation of the regulating dam at the downstream of the main dam, this equation started to give 2% – 3% errors due to the daily fluctuation of the regulating pond. The following modified equation was then used to eliminate these errors.

$$Upsteam\ EFF\ \% = \frac{U/Sp - U/FL}{R.L - U/FL} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\text{(iii)}$$

In this Equation U/Sp and R.L are same as above, and;

U/FL is the level of the piezometer filter in the upstream piezometer

This equation gives the efficiency of the curtain in its upstream part and the results should vary between 50% to 100%.

The efficiency of the curtain in its downstream part is given by:

$$Down\ stream\ EFF\ \% = \frac{U/Sp - D/FL}{R.L - D/FL} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ \text{(iv)}$$

D/FL is the level of the piezometer filter in the downstream piezometer.

The result can vary between 0% and 50%.

It is interesting to note that the Mosul Dam International Board of Experts (IBOE) formed of three top world dam experts was following the design and construction of Mosul Dam since 1979 and this also included the entire grouting curtain works. The board approved the use of equation (ii) since 1987. The following interesting comment on this subject is taken from IBO report:

"Of particular informational value was the ratio or index computed by the engineer to facilitate rapid appraisal and especially to get comparisons of situations at the same station with passage of time and comparisons of various stations. The index gives a measure of difference in head between upstream and downstream piezometric pairs divided by the difference between the headwater and tail water. The resulting number ranges, in practical examples from 0.2 up to 0.9. The higher the index is the higher the effectiveness of the curtain in that location, with values of 0.3 to 0.4 and lower signaling a possible need for remedial action. In addition, at the same location, a gradual lowering of the index with passage of time signifies a deterioration of the curtain with need for remedial action" (IBOE, 1988a).

In another IBOE report the following recommendation was made: -

"100% efficiency is not possible and 70% is well within acceptability limit and repair should be done if efficiency goes below 50 %"(IBOE, 1988b).

Continuation of the piezometric observation resulted in the repeated application of both normal and massive grouting works up to now as a repair measure. The location of the major grout takes between sections 69 and 84 is shown in figure 2.

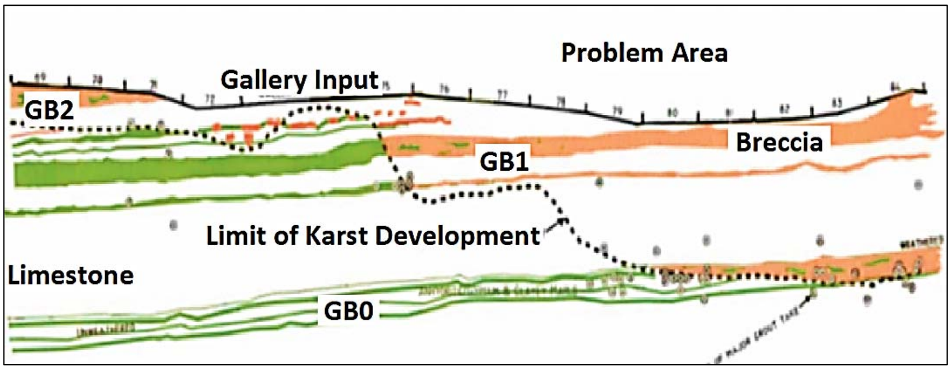

Figure 2: Major Grout takes in sections 69-84.

There are other high grout take zones along the dam but this area appears to be one of major concern. The heavy line shown on the crest of the dam in figure 2 shows the approximate extent of this problem (high grout take) area. The green layers represent anhydrite while the orange represent gypsum breccia (nodules). The major grout takes are located close to the karst line associated with the conversion of anhydrite to gypsum (and also marking of the change from a high permeability to a lower permeability foundation), and also in the gypsum breccia (nodules). The white area between the green and orange layers consists of karstified limestone. It often contains voids, fractures, and fissures (International Washington Group and Black and Veatch, 2005).

5. Future Prospects of Massive Grouting

The massive grouting technique has so far proved its usefulness in sustaining the dissolution process into manageable condition using the present working capacities available at the dam site. Massive grouting; however, has not managed yet in terminating the dissolution processes altogether and it seems that it is not likely to do so in the future. The continuation of this program, year after year does not preclude some bad implications and it is of very high cost. Other consequences of a worst nature may be also considered. Today, the future consequences of Massive grouting cannot be guessed or quantified and many scenarios can be debated. The worst of such scenarios is the deterioration of an extensive mass of gypsum/anhydrite layer close to the curtain and changing into a skeletal structure; as a result of repeated injection of grout mixes. Such a structure could collapse under increasing stress leaving big cavity or cavities behind. Concentrated seepage flow will then attack what remains of the curtain washing large areas of it. The invigorated solution and collapsing process that follows can progress very rapidly and create a situation, which exceeds the present available technical capacity of massive grouting. In a runaway situation, like this where the rapid deterioration of the foundation continues; a downward sinking of the dam can occur once the layers close to the dam base are affected.

The sudden failure of Mosul Dam will result in a huge catastrophe. Massive grouting can be looked at as a temporary solution that may delay the occurrence of such an event, but it will not give the final solution. Such solutions have been discussed for the past thirty years or so.

At the time being, research work is required to improve massive grout durability by adding chemicals, which may interact with the gypsum and form micro films around them and hinder dissolution. This can help to improve the gypsum resistance and increase its stability. Mathematical models may be also studied to understand the mechanism of cavities formation and collapsing to have a clear picture of the consequence of such processes.

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CHAPTER 7: EXPERTS PROPOSALS AND IDEAS ON MOSUL DAM

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1. Introduction

During early 50's, Iraq embarked on a very ambitious program of development including advancing the agricultural sector. This included the extension of irrigated areas in large swaths in the north, middle and south of the country. Constructing dams on the river Tigris and its tributaries was required to provide the extra water needed. Constructing dams on the river Tigris and its tributaries was required to provide the extra water needed.

The construction of Mosul Dam was; therefore, contemplated in 1951; investigations and studies were ordered then. Various studies and reports were conducted during the following 30 years by many specialized international firms covering various locations upstream of Mosul city and providing different alternative designs.

All the studies expressed a clear concern on the fact that this region suffers from extensive presence of soluble rock formations that might undermine the safety of a high dam of a large reservoir such as Mosul Dam. A very refined study that was carried out later by a Baghdad University team (Al-Ansari et al., 1984) on a very large area surrounding the reservoir showed a lot of anomalies and karstic solution phenomena. Having known these facts before, could have affected the decision on the selection of the present location of the dam or even introducing the idea of building a series of smaller dams in this reach of the river. This study utilized air photos, satellite imagery and magneto metric measurements, which were not in use during the planning stage of Mosul Dam. Such methods could be utilized in planning of future dams in areas of similar nature.

2. During Construction and Impoundment

Extensive amount of literature is available on Mosul Dam at this period comprised of the writings of experts involved in the construction activates of the dam or visiting as expert panels to review these activities or even commissioned to find solution to a particular problem. Foundation problems took prominence among these writings. From the reports of the International Board of Experts for Mosul Dam it is clear that the Board was worried on the nature of the dam foundations since its first meeting and through to the end of construction. In more than thirty reports, the nature of this foundation, foundation treatment and safety hazards due to the dissolution of gypsum and anhydrite took the bulk of those reports. It is worth to mention that this Board was formed by The Ministry of Irrigation of three top world dam experts to follow the planning, design and construction of the dam.

In the Board first report on its review of the draft planning report prepared by the designers in 1979, it states the following:-

"*The presence of gypsum and anhydrite in the marl would be particularly detrimental in zones where high seepage flows could develop, i.e. mainly at the contact with cavernous limestone. This will require a special attention in avoiding high velocities, induced by high hydraulic gradients", and it even goes on to recommend allowing ample cost in the cost estimate for foundation treatment as it was very difficult to come up with a good estimate in such difficult rock conditions as in Mosul Dam*" (IBOE,1979).

It must be remembered that, this report was written while geological investigations were going on and a clear picture of the behavior of rock with grouting was not established yet. One area of major interest was the treatment of the dam foundations, the work on which had started in 1982. This was initiated by drilling deep exploratory holes and performing pressure water tests to define the details of the grouting program. The work was continued by the grouting contractor's engineers, the designer's engineers with the constant following and suggestions of the IBOE to come up with the required Method Statement answering for both blanket and deep curtain grouting. This included the definition of the acceptance criteria of the finished work, techniques and methods and sequences of implementation. As expected, many difficulties were encountered in the deep grouting part, especially at the chalky series over GB layers and in Jeribe Formations. The years from 1985 to 1987 and 1988 were full with surprises and setbacks. The seriousness of the matter was magnified by the rising levels of the reservoir after river closure in 1985.This situation called for the advice and guidance from grouting experts, specialized grouting and engineering firms. Of these it is worth mentioning the report of (Luga, 1986), which was concentrated on examining the quality of the completed blanket grouting, the right bank curtain, the left bank curtain and the ongoing work on the left extension curtain. It also gave recommendation on the use of silica gel for the grouting of the left bank curtain in order to reduce seepage appearing in the left bank.

Another important study concentrated on the grouting of the deep grout curtain under the main dam. This study was completed in order to assist the discussion of the problem of achieving a satisfactory dam foundation (Binnie and Parteners, 1987).The study report explored the very high take of cement grouts and the sand and gravel mixes in a number of zones causing concern. Solution channels and cavities were developing at such rate that the grouting program was not capable of maintaining an adequate curtain. The question raised was whether the dissolution in gypsum/anhydrite beds was taking place at a faster rate than the sealing effect of grouting processes. A theoretical analysis of enlargement of small passage within a soluble rock, which was included showed that a flow in a pipe of dia. 60 mm (60 mm is chosen as 3x19 mm, the maximum size of gravel used) the flow rate was calculated as 0.0028 m^3/s, a velocity about 1 m/s, the calculation showed that the pipe enlarges to a diameter of 125 mm in a year. It also follows from the study that the equivalent pipe volume is proportional to the cube of the solution potential, so that if the concentration of sulfate is for example 500 mg/l instead of the 750 mg/l assumed in the analysis representing the typical concentration opposite problematic sections, then the pipe would enlarge from 60 mm to 300 mm. As a conclusion, the dissolution that had occurred since impounding was substantial, but not so great as to preclude a successful completion.

The study continues to suggest methods to be used for attaining adequate treatment by using gelling cement grouts or trying the injection of hot bitumen or diesel oil and bentonite mixture or even the addition of cotton flock or similar fibrous material (such mixture was used to staunch high velocity flow at Dokan dam in 1960's in Iraq). Other alternatives involving in the construction of physical barriers were also examined.
Recognizing the two basic engineering problems;

a) Pre-existing karstified ground at a depth as great as 110 m.
b) Flowing water at these depths capable of dissolving gypsum beds, which may be enlarging the flow paths.

Then; admitting the fact that, high grout takes associated with large channels had been found between Sec.75 and Sec.93; the study examined the possibility of emptying the reservoir to relief the foundation from the existing artesian pressure to enable a better performance of grouting operations. This was overruled since the curtain should later resist even greater pressure at full impoundment of the reservoir. So this led to another line of thinking by considering three basic approaches;

a) General filing of seepage paths by precipitation of insoluble materials from ground seepage water. This envisages the fact that injecting a fluid such as of sodium chloride and utilizing common ion effect then insoluble precipitant would fill and close the seepage paths. This was rejected by the fact that the precipitant volume will not be enough to fill seepage paths of various sizes plus the other fact that the injected fluid may flow in an unpredicted manner by varying seepage flows.
b) Sealing gypsum/anhydrite surface; processes which can protect gypsum or anhydrite surfaces against dissolution and serve as valuable aid as seepage control measure. There are two approaches; the first is by maintaining a saturated or super saturated solution with SO_4 against the rock surfaces, which requires a continuous supply of solution to seepage water probably from an upstream gypsum blanket. The indicated quantities of removed gypsum daily from the foundation and the uncertainty of directions taken by seepage flow path make this an unattractive proposition. The second is to provide a chemical solution, which react with $CaSO_4$ to form a highly insoluble coating. “Washing” the calcium sulfate rock surfaces within the seepage passage with such a solution might line them with a protective coating against further dissolution. This idea suggests using calcium oxalate, but the problem facing this solution is the tenacity and durability of such coating against further seepage and the uncertainty and difficulties of judging the performance with time. There are also the known toxic effects of oxalate to animals and humans and the large volumes required to be released in ground water. All those give a strong argument against this solution also.
c) Barriers.
   i) Blanketing

Blanketing of the dam and reservoir is illustrated in figure 1. Effective blanket would affect the piezometric profile in the foundations. Impervious upstream blankets would normally be installed only by drawing down the reservoir. Their success depends on the entry points of seepage to the foundations and lengthening of seepage paths.

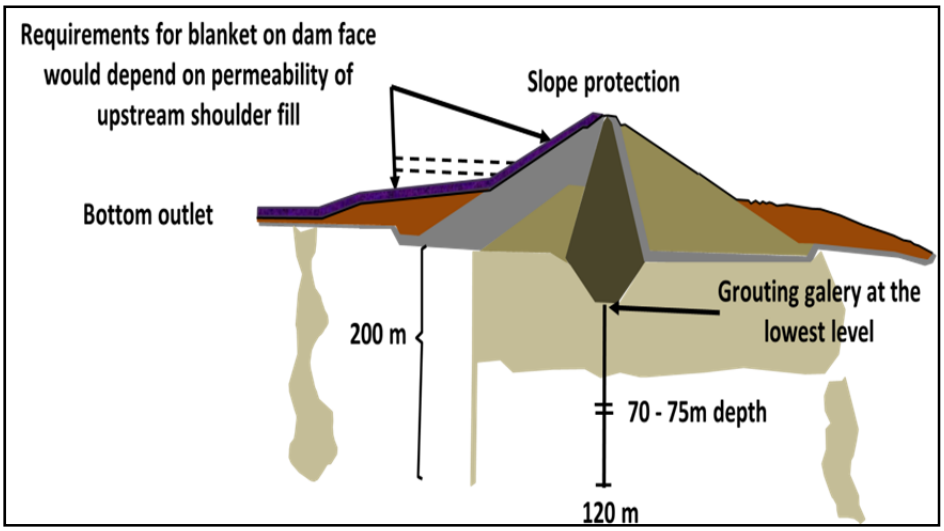

Figure 1: Illustration of upstream blanketing arrangement.

Blanketing the bed can be done without drawing down the reservoir but by dropping the lining materials through pipes lowered to the bottom of the pond. Bentonite pellets are dropped in place first, and as bentonite absorbs water a highly impervious layer is formed. Sand is placed on top to hold the clay in place against any disturbances. But, as far as it is known this method has been used for small ponds and not for a reservoir of this size.

ii) Positive-Cutoffs

The use of cutoffs is illustrated in figure 2. Difficulties in using this method result from the combination of depth and hardness of strata between the pervious surface and the impervious bed beneath the karstification zone. This may make trenching operations very problematic. The depth of the required trench has a great effect on the cost. A location near the upstream toe of the dam would require the least depth but it requires the drawing down of the reservoir. A trench from the crest through the core is a tricky operation and it extends for a considerable depth into the foundation, which may be technologically questionable and very costly.

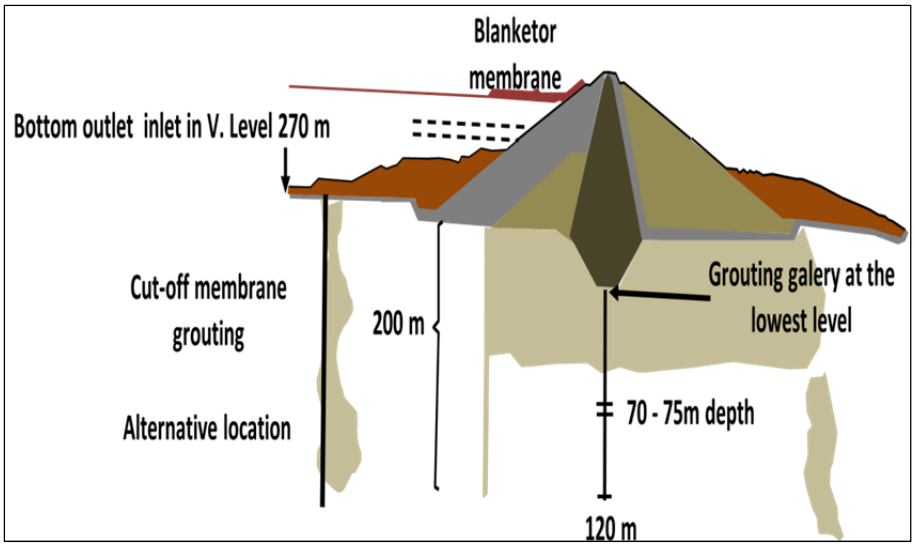

Figure 2: Illustration of upstream cut-off arrangement.

iii) Construction of a new curtain.

If new curtain is constructed as a direct reinforcement to the present one, this assumes it should be done in a better geological location in the upstream direction of the dam if it would provide any improvement. But, such better geology does not exist and because the construction requires the drawing down of the reservoir then such proposal has no value.

This study showed that all the discussed alternatives were not practical and some of them were even not feasible. The logical conclusion was to continue the works on the present curtain by improving the mixes and injection procedure to combat large take areas, sealing large pipes and channels, providing a new array of piezometers taping the known solution areas especially the contact between the pervious limestone and GB0 layers to monitor the efficiency and the long term performance of the curtain in these soluble layers where windows were most likely would develop.

A new concept was formulated by the study; that is a satisfactory cut-off may be deemed either as one which had been so tightly grouted as to allow negligible dissolution in the long term or one in which dissolution may take place but is kept under control by comprehensive, appropriate and effective maintenance grouting. Means to facilitate and guide maintenance grouting work must be developed.

The three major consequences of the study were:

1. It gave a new dynamics to the "groutability test program" described in the previous section.
2. It introduced the concept of "Maintenance Grouting" as a long term safety procedure.
3. It emphasized the importance of piezometers as a mean of checking local solution areas for prioritizing the repair works.

At the same time, two more studies were made available to the IBOE. The first one was presented by Dr. Aladdin Hamdi from Mosul University who had performed basic research on gypsum and anhydrite. The work covered:

-Physical properties with particular attention to mechanical strength criteria.

-Dynamic properties with sonic properties and changes after impounding of reservoir.

-Petrographic properties, normal and changes.

-Creep under load.An interesting aspect was the changes in stress-strain relationship to anhydrite, the effect of increasing porosity (by solution) on sonic wave velocities leading to relationships of sonic properties versus percentage of solubility. In situ condition was simulated by tri axial testing under high confining pressure (Hamdi, 1987). The importance of this study stems from the fact that it represented a forerunner of needed knowledge, especially the way the strength qualities of both gypsum and anhydrite rock layers deteriorates at depth leaving behind much doubt about their stability and their susceptibility to collapse.

The second study presented to the Board was concerned with the grouting of problematic areas in the gypsum/anhydrite beds(Alrawi,1987).The prescribed procedure was to replace sand-gravel mixture in the cement grout mix by heavier material that would readily settle in low spots during conditions of turbulent flow. The suggested materials were barite ($BaSO_4$) with a specific

gravity of 4.2 - 4.5 or hematite (Fe_2O_3) with specific gravity of 5.0 - 5.2. It was also suggested to use 15 mm diameter grout holes to place large volumes of this grout. The advantages of this method were discussed and agreed upon, but this meant the procurement of new equipment such as heavy drilling rigs, new casings…. etc., which would entail additional costs and require much time to deliver.

With the continuation of repeated efforts to keep the integrity of the curtain using massive grouting, the question of looking for other alternatives remained a constant worry to the owner (Ministry of Irrigation). The ministry called for the help of another specialist (Mariotti) to examine the condition of the curtain and give suggestions. Mr. Mariotti's report was submitted to the IBOE at the end of 1988, the following proposals are found with the Board commentary (IBOE, 1989):

a) In the context of strengthening the grout curtain in the problem areas where massive grouting had to be repeated widening the curtain was recommended. Additional rows of boreholes ought to be drilled consisting of one row upstream of the present curtain and slightly inclined towards the upstream, another row in the downstream of the present curtain and inclined towards downstream, and finally a central vertical row in between. The central row was to be grouted first followed by the upstream row and then the downstream. Finally, the central row would be re-drilled and fine grouting to be performed using silica gel .The Board did not object to this proposal as machinery and grouting capacity were available.

b) The second solution was to construct a tunnel the length of the chalky series from which grouting would be performed. The Board thought that such work was very specialized and would need an expert studies to check its feasibility.

c) The third solution was to construct a series of tunnels and galleries to replace risky material. This alternative received the same comments as in (b) above.

d) The fourth alternative was to construct a diaphragm wall from the upstream berm, with a sloping concrete facing from the top of diaphragm to the top of the dam, Fig (3). Or even to remove part of the top of the dam and install the diaphragm through the core in a location upstream of the gallery. This arrangement was attributed to the unavailability of machines that could cut to the desired level. In a later update, the removal of the dam top was thought unnecessary due to new development in diaphragm machines. The Board; however, judged this solution undesirable due to the required lowering of the reservoir level, which could extend 2-3 years in addition to the very high cost.

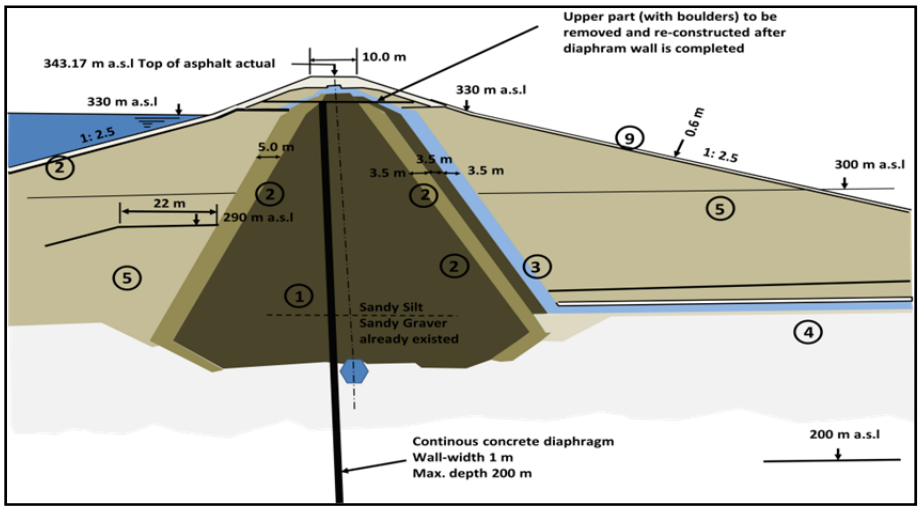

Figure 3: Proposed Diaphragm driven from the dam crest.

3. Badush Dam

The years 1986-1988 were years of uncertainty and worries over the Mosul Dam foundation. The catastrophic consequences of the dam failure were already explored and quantified by the "Mosul Dam Flood Wave Study "(Swiss Consultants, 1984). The Ministry of Irrigation was advised then to take some protective measures to secure the safety of the downstream area and its' population. The design and construction of Badush Dam was initiated in 1988 using fast track method to complete the dam within four years.

The Badush Dam site is located on the Tigris River, approximately 40 km downstream from Mosul Dam site and approximately 15 km upstream of Mosul city, Fig (4).

The main function of the dam was protection of the downstream region of the Tigris River valley against the effect of potential Mosul Dam failure, due to any possible reason. Other functions were included to add other benefits in order to improve the economic feasibility of the project. These benefits were namely power generation, by using water discharged by Mosul Dam and by the regulating scheme power plants. Badush Dam power station was to have an installed capacity of 170 Mw. Therefore, about 15 m head is provided for Badush hydroelectric power plant and for normal regime flood protection (up to 1/10000 year flood). The Badush Dam could guarantee the operation of the Mosul power plant as well as of the power plant of the regulating scheme by discharging variable quantities of water, limited to 8000 $m^3.sec^{-1}$ for the safety of Mosul City (Energoproject, 1988).

The design of the dam allowed a free volume of 61.5 m height between EL. 245.4 m (a.s.l.), which is the normal operation water level, and EL. 307.0 m (a.s.l.), which is the maximum water level in case of Mosul Dam failure. This volume is enough to contain the flood wave resulting from the worst scenario of the Mosul Dam flood wave study. The foundation of Badush Dam was of massive limestone and no dissolution of soluble rock was anticipated.

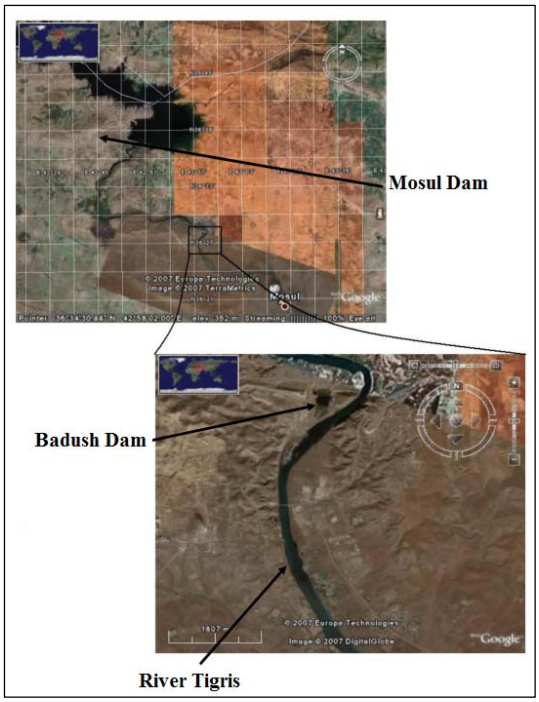

Figure 4: Badush Dam location in relation to Mosul Dam (Al-Adily et al., 2014).

The construction of the dam began in 1988 at the time when no final solution was in sight for Mosul Dam. The construction however was halted in 1991 due to the economic sanctions that were imposed on Iraq as a direct result of Iraqis' occupation of Kuwait. The percentage of completed works is 40%, and the resumption of this work is waiting the decision of the Ministry of Water Resources (Previously the Ministry of Irrigation). It is disappointing to know that the ministry has revised the design of the dam in 2007 by lowering the maximum pool water level and the height of the dam so as to eliminate the flood wave protection.

4. Studies Carried out During the Operation of the Dam

While the maintenance grouting continued during 1986 and the following years using both normal mixes and massive mixes, consumption of both types and the total consumption were sizable indicating continuous washing away of some areas of the curtain. No general review of the dam safety was carried out until 1995. A general inspection of the dam and a review of all the available reports and recorded measurement were conducted by two Bulgarian specialists who stayed for two months at the site and then submitted their report with their findings (Markov and Dimitrov, 1995). The report covered all aspects of the dam performance that far, but it did not attempt to analyze the grouting process implications on the dam foundation. It is useful however to summarize here conclusions and recommendations:

i) Observing the area in right bank at 200m upstream of the dam there was a big crack which indicated the movement of a sliding block. The observation system looked good and regular measurements were recorded. It was necessary to continue this logging.
ii) At a distance of 2 - 3 km from the main dam in the upstream right bank there were many subsiding planes indicating the formation processes of sinkholes. The belief was expressed that such sinkholes were contributing to seepage under the dam at the higher levels of the

reservoir. It is worth mentioning here that these and other sinkholes have been discussed at length during many IOBE meetings but no firm conclusions were formed then.

iii) The width of the deep grout curtain was checked according to Russian code No.СН-ИП 2.02.85, 1988 table7, and the Bulgarian code No.2.07.03, 1985. The check results showed that this width was on the minimal side and the curtain requires widening at depth, therefore it was recommended to drill a new row of inclined grout holes at the back of the downstream row of the present curtain in order to strengthen it.

iv) Checking of piezometric observations revealed that 90% of the piezometers were giving good results, while the remaining 10% of the piezometers were giving erroneous results and need to be flushed.

v) The piezometric observation records of all piezometers located in the grouting gallery from Sec. 78 to Sec. 94, and calculating efficiencies using the same efficiency equation that had been used by the Swiss Consultants, showed efficiency values averaging between 65% to 70% at a maximum reservoir level of 328 m (a.s.l.), while lower values were recorded under maximum water level of only 310 m (a.s.l.) in the years 1987 and 1988 .The conclusion was that the curtain was judged good. In this argument this report overlooked the fact that average values over this long reach could not give judgment on the whole length especially that minimum efficiencies as low as 42% were also experienced in that year in local spots. The report also did not mention the fact that almost 43000 tons of solids were injected in the grout curtain during the years 1988-1995.

vi) The seepage quantities and soluble salts' concentrations recorded in the previous period from 1986 up to 1995 showed great reduction in seepage quantity and salts concentrations at point no.1. This was affected by the construction of a deep grout curtain during 1990 of 500 m length parallel to and at 10m distance from the spillway chute. At point No. 3 seepage quantity had increased between 1985 to 1995 from 75 l/s to 140 l/s and salt concentrations rose between 1990 and 1995 from 800 mg/l to 1200 mg/l which was attributed to the rise of water level of the reservoir. Seepage quantities and soluble salts' concentrations from the access tunnel remained constant at their previous level in 1985.

vii) Checking the results of the extensometers placed in the body of the dam and the geodetic measurements for the dam at the crest showed that the displacements were within acceptable limits. Similar conclusion was drawn regarding the grouting gallery. Pore pressure cells gave also normal readings for the dam core and dam shell at the locations where they were installed.

In the general conclusions and recommendations the study goes on to state the following;

i) The present state of the dam (1995) is judged to be good, but the operation of the reservoir should not attempt impoundment at level 330 m (a.s.l.) only in emergencies and for a very short time.

ii) While total seepage quantities and dissolved salts in point no.1 are reduced due to the completion of grout curtain parallel to the spillways' chute ,these quantities in the other measuring points seem to increase at high reservoir water levels.

iii) The analysis of the recorded data of the functioning extensometers, pore pressure cells, geodetic measurements shows acceptable values.

iv) Additional rows of grouting holes as described above are required for the reasons given. But, the construction of a diaphragm wall as proposed by Mariotti-GEOCONSEL was rejected.
v) The Maintenance grouting program of the deep grout curtain, started in 1990 (in fact 1988) should continue. This program may be needed for the whole operating life of the dam.
vi) More piezometers are required to be installed along about 2000 meter downstream shell of the dam in order to observe seepage in the dam foundation.
vii) Performance of a new static and dynamic analysis of the dam is required by using updated information (water levels, Seepage rates and quantities, vertical displacements of the dam body, pore pressure measurements in the dam body).

During the following years no rigorous evaluation of the dam safety was performed. Only annual reports of the recorded data of the instruments, piezometric observations, reservoir water levels, water releases and monthly consumption of grout mixes and locations of treated sections were presented.

In 2003, Iraq was occupied by the coalition forces and a site visit by USAC team was made in order to assess the magnitude of the threat that the dam posed to the occupation forces. This was followed by the signing of a contract with WII/JandB.JV, in order to carry out thorough engineering evaluation of the existing problems in need of correction, define potential and alternative solutions and identify and come up with concrete recommendations to improve them. The scope of work was to carry out an initial assessment with site visits and submission of a preliminary report, collecting and documenting all available studies, designs and reports, formation of a panel of top geotechnical experts POE to study the foundation treatment and presenting solutions in a final report.

The initial report produced under the contract (Wheeler, 2004) gave an overall picture of the geology of the site and foundation conditions, grouting activities, current seepage conditions, and description of the sinkhole phenomena. The report was concluded by some general analysis and recommendations for further discussion by the POE, which was already formed of four geotechnical experts. The activity of collecting all available data on the dam was also going on at that time and took some considerable time and effort. Some 10500 pages of documents were compiled and recorded on CD's. These data were collected from the Dam Management office, Dams and Reservoir office in Baghdad, Consultants' office in Switzerland and other Consultants offices in Britain. This comprehensive library was put at the disposal of the POE who continued its discussions and meetings.

In the following deliberations the Panel concentrated on the study of seepage and dissolution of gypsum/anhydrite beds. The panel confirmed that the "Enhanced Grouting Program "of the grout curtain maintenance was fully justified considering the annualized probability of dam failure. But, this program would not reduce this probability to acceptable limits. According to the (USBR) guidelines, the annualized probability of risk when human lives are at stake should not be higher than 10^{-2}. Although this annualized risk would be improved considerably by

continuation of this maintenance program, it will not reach with the foundation to such acceptable annualized risk.

In viewing the current program, the Panel could not overlook some of the positive aspects, as progressive benefits were being achieved, e.g. reduction in the frequency of sand-cement injection, general reduction in average grout consumption, favorable piezometric response….etc. In addition, the training of an efficient team of experienced work force was looked at as a good asset. In the foregoing, the Panel confirmed the appropriateness of introducing "New Technologies" in grouting techniques of the "IntelliGrout" systems was mentioned.

The panel did not fail to study the feasibility of the construction of a concrete wall as a positive cutoff, which was done in great details. The Panel however rejected this proposal for the following reasons;

a) Maximum depth attempted on any existing dam was 130 m at that time. Only one case of 150 m was recorded.
b) Mechanical Limitations, wall panel deviation and the foreseeable task of having to retrieve stuck equipment from great depth render the prospects of a wall deeper than 130 m practically impossible.
c) The maximum unconfined compressive strength of rock material which is possible to excavate with a wall cutting machine may be as high as 200 MPa in a rock masses, which are highly fractured or thinly bedded. However, the limit will be much lower if rock is massive (perhaps 70M Pa).
d) Construction under full reservoir conditions is feasible if the slurry can be maintained at least 1 m above lake elevation continuously throughout the construction of each individual panel.
e) When there is potential for loss or lateral movement of slurry during construction, then pre-grouting of rock masses is necessary (as would be the case in Mosul Dam).
f) Such a cutoff wall cannot be built from a small gallery, which is subject to hydrostatic head as is the case at Mosul Dam.
g) Deep diaphragm walls in rock (when constructible) would cost on the order of $3000 per square meter (price levels of 2005) and only 3 square meters per hour per machine could be anticipated.

It was understood by the Panel that the reservoir could not be drawn down below EL 306 m (a.s.l.) for any significant period of time due to the power station operating limitations. This eliminates the possibility of construction of such wall from a reduced starting elevation on the upstream berm of the dam where water depth would be less. The impermeable structural connection details for connecting the top and ends of the cutoff wall to the dam core would present a design challenge that could well be impossible to resolve.

In support of its decision of rejecting the diaphragm alternative, the Panel, explained that this rejection also stems from the following considerations:

-Would be extremely costly and with severe technological limitations.
-Would take a great deal of time and resources.

-May not be geometrically feasible (since the pre-treatment would have to project upstream, which is not possible from the gallery) as the vertical wall would have to avoid the gallery.

Furthermore, the potentially adverse effects on modified water movement pattern in the foundation that may result from creating a virtually impermeable concrete wall of an undetermined depth and length is not known, and may not be identified with sufficient precision, to date, the seepage rates and location are not known. Such information is essential for the contemplation of a shallow "hanging" wall and only penetrating 30 m in the impermeable rock. In considering the previous suggestion of using hot bitumen for the injection works, the Panel explained that such work would demand the use of highly specialized equipment and require a new set of facilities, such as specialized bitumen storage, pumping and delivery infra-structure that may not be logistically possible in a gallery situation and when dealing with high artesian heads . Based on these the Panel recommended that no further consideration be given to the use of this material in the gallery of Mosul Dam.

Sinkholes were the other area of interest of the Panel, which had received its share of attention and consideration as one dangerous threat to the dam. The mode of formation of the previous sinkholes and their locations was studied in relation to their geological setting. The Panel did not possess; however, any data whether the sinkholes were formed under the downstream shell of the dam or not. The prediction of sinkholes formation as well as the solution conduits beneath the embankment was obviously, and still is, a critical issue. But the technical options are in reality extremely limited and much would depend on the implementation of the modified drilling and grouting processes as well as the continuing analysis of data from existing, and new, piezometers.

A great bulk of data on the monitoring of instruments and observation systems were examined. But the opinion of the Panel was that efforts should concentrate more on the critical family of systems such as the geodetic survey system on the crest of the dam and in the gallery, Piezometers in the downstream shell of the dam and in the gallery, and on the seepage measurement weirs located at the downstream left bank. The conclusion was that the monitoring program was in need of re-evaluation and revision. Concentration should be given to the aspects just mentioned. More piezometers should be installed in the downstream shell of the dam. They should aim at the gypsum/anhydrite layers directly under the dam and other layers which had experienced considerable grouting takes in shallow depths. The present number of piezometer in the whole area of concern downstream of the dam was minimal. For in this area, of about 500000 square meters only I piezometer in 11 acres (about 40000 sq. meters) was provided, this is not enough to give a clear picture of all seepage paths existing or in the process of development there. In the river channel downstream the dam no more seepage points could be observed on the surface as the presence of such points would be masked by the existing re-regulating pond downstream this leaves another unknown to the seepage picture.

Two other safety issues were also given attention; the first one was the magnitude of scour at the bottom outlets Roll-Crete lining of the plunge pool resulting from the operation of these bottom outlets at high floods. This was causing worry about the integrity of the foundation of the

bottom outlets downstream control structure. Mathematical analysis of this hydraulic problem was carried out and the results showed that such erosion would not cause a threat to the stability of the outlets control structure foundations, especially that the analysis gave similar results to the hydraulic model test results during the design stage. No protection using H-piles were needed as though previously.

The second question was the stability of the spillway chute. Large seepage quantities were reported in the spillway bucket area in 1986 and in the following years. It was thought then that part of this flow was coming under the spillway chute from the right side into the left side. This structure was founded partly on the gypsum layer GB3.The seepage quantities were considerably reduced after the completion of the grout curtain parallel to the chute and along its length in 1990. The question of the foundation integrity remained a worrying question. Considering this matter the Panel observed that not enough data was available at hand to make a meaningful assessment, and recommended the following to be done in the future;

- To study the historic data from preconstruction through to current day operation of the spillway including all exploration bore holes and grouting records.
- Conduct field observation and analyze flow measurements with respect to lake levels.
- Review all survey data on the structure to detect any permanent deformation or settlement.
- Perform crack surveys of the concrete structure itself.
- Assimilate and analyze the data with the objective of determining whether corrective measures should be initiated to reduce the leakage, or deciding that the threat to structural integrity of the chute requires remedies by drilling and grouting techniques (WII/JV, 2005).

In summary, this study has proved so far as the most through and comprehensive one carried on Mosul Dam conditions. The stated recommendations were very clear and realistic. Among these recommendations, the completion of Badush Dam was considered the only long term solution that can provide acceptable level of risk to the population downstream that could result from Mosul Dam failure.

In this respect, it is worth to mention here that the Ministry of Water Resources of the Iraqi Government has committed a grave mistake in two unfortunate steps in disregard to this recommendation; namely the redesign of Badush dam as a low dam eliminating its large protective storage volume, and signing in 2006 a letter of understanding with one German company (Pipes, 2011) to build diaphragm machine(s) and construct a diaphragm wall at a cost of 2.6 billion US dollars. Signing the contracts awaits the availability of funds. Both decisions are reversible and the Ministry must be brought to the understanding to where the best interests of the Iraqi people lies.

In late 2005 The Ministry of Water Resources commissioned another Board of Experts to carry out an Independent evaluation of the dam, but the IIW/BandV report and their Panel study was so comprehensive that this new Board could not come up with any new fundamental finding. The worry about the seepage under the dam was the same and led this Board to advise on increasing the number of piezometers in the same way that was advised before and to carry

out a geo-radar survey in the left bank to try understand the seepage water regime. This was done and all required instruments were purchased and site geologists were trained to carry out such surveys. The results were not conclusive and the Board decided to limit the maximum operation water level to EL 319 m (a.s.l.) instead of EL 330 m (a.s.l.).
This recommendation has remained in force up to now with the loss of sizable storage of irrigation water and power potential.

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CHAPTER 8: DAM FAILURE AND ITS CONSEQUENCES

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1. Introduction

Dams are very important infrastructure to any country. They serve for different purposes, e.g. flood control, water supply, hydropower generation, irrigation, navigation and recreation benefits. Unfortunately, these huge structures represent risks to life and property due to their potential to fail and cause catastrophic flooding (Wahl, 2010). There are many dam failure events that were caused due to different reasons. For details of dam failure see US Department of Homeland Security (2011), Colorado Dam Safety Branch (2010). Mitigation of such risks requires continuous inspection and maintenance of all dams. In studying dam failure, the prediction of the reservoir outflow hydrograph and the routing of the hydrograph through downstream valley to determine the dam failure consequences are to be carried out. Usually, the prediction of the reservoir hydrograph is uncertain; in particular for embankment dams where the dam failure is due to progressive erosion processes that are complex and difficult to model (Wahl, 2010). Wahl (2010) reviewed the modeling strategies of dam failure and summarized them (see table1).

Table 1: Dam break flood modeling strategies.
The first column indicates different approaches to determine breach parameters and/ or the breach outflow hydrograph (Wahl, 2010).

<table>
<tr><td>Regression model for Qp as function of dam and reservoir properties</td><td rowspan="2">Approximate breach outflow hydrograph by predicting peak outflow and hydrograph shape directly.</td><td rowspan="5">Route breach outflow hydrograph to determine flooding consequences</td></tr>
<tr><td>Analytical model to predict Qp with closed form equations or charts as functions of dam and reservoir properties</td></tr>
<tr><td>Regression model for breach parameters as functions of dam and reservoir properties</td><td rowspan="2">Provide breach parameters as input to routing model, which determines breach outflow hydrograph by the use of hydraulic equations for flow through enlarging breach</td></tr>
<tr><td>Apply erosion model to predict breach evolution and the approximate breach description in parametric way for input to routing model</td></tr>
<tr><td colspan="2">Process- based erosion and hydraulic models that simultaneously determine breach development and resulting outflow hydrograph</td></tr>
</table>

The details of the regression models listed in table 1 can be obtained as follow:

1. Regression Models for Peak Outflow: SCS (1981); MacDonald and Langridge- Monopolis (1984); Costa (1985).
2. Analytical Models to Predict Peak Outflow: Cristofano (1965); Fread (1988).
3. Regression Models for Breach Parameters: Wahl (2004); Froehlich (2008).
4. Erosion Models Leading to Parametric Breach Descriptions: Fread (1988).

5. Process-Based Dam Breach Models Integrated with Dam-Break Flood Routing: Mohamed (2002); Temple et al. (2005).

Dam failure is closely related to foundation and/ or spillway problems (ICOLD, 1974). Leakage in the foundation and embankments are the major reasons in the incidents of earth fill, and earth and rock fill dams (ICOLD, 1974). Foundation incidents are related to improper interpretation of the geology of the site and improper treatment, while seepage from embankments is usually due to poor construction work.

Construction work in Mosul Dam started on January 25th, 1981 and started operating on 24th July, 1986 (Fig. 1). The dam was constructed on highly karstified beds of the Fatha Formation. In view of this fact, grouting operations were conducted during the construction period to fill the cavities, fissures, joints and cracks in the karstified beds. Unfortunately, all the executed efforts did not stop the seepage under the foundation of the dam. After impounding in 1986, new seepage locations were recognized. Grouting operations continued and various studies were conducted to find suitable grout or technique to overcome this problem. The seepage due to the dissolution of gypsum and anhydrite beds raised big concern about the safety of the dam and its possible failure. US Army Corps of Engineers conducted a study on Mosul Dam for the period June, 2004 to July, 2007 and highlighted the possibility of the dam failure (Wakeley et al., 2007). News media had highlighted this concern in 2014 when ISIS occupied the dam site area (RT, 2014; Roulo, 2014; The Independent, 2014; Bender, 2014; Tomkiw, 2014).

Figure 1: Mosul Dam,

2. Swiss Consultants Dam Failure Model Used:

Swiss Consultants (1984) (SC) carried out a comprehensive study for the period 1983 – 1984 on the possibilities of Mosul Dam failure using FLORIS model. They investigated the consequences of a hypothetical failure of the Mosul Dam. The report highlights the dimensions of a disaster, which could occur if the dam maintenance and protection were carelessly neglected, thus underlining the importance of the careful dam safety monitoring. The study included a summary of dam failure assumptions, initial wave calculations and results of flood routing.

The study was presented in three volumes; it includes five chapters, where in chapter 2 the mathematical model for Tigris River was described. This included the description of the model theory, the schematization of the river channel downstream of the dam and the representation of Mosul reservoir. In chapter 3, calibration of the mathematical model for the Tigris River was

discussed and the calibration of the model was based on reproducing historic floods. In chapter 4, dam break scenario and initial wave calculations were explained. The assumptions surrounding the mechanism of dam failure and its representation in the mathematical model were described. Finally, in chapter 5, results of flood routing were given. This includes discharge and water level profiles to beyond Baghdad. The magnitude of (1000 – 10000) years flood and the PMF (Probable Maximum Flood), together with information on the discharge capacities of the main and fuse-plug spillways were provided in the study (Table 2). Furthermore, the report includes information about the spillway capacities in the event of greater degrees of inoperability of the main spillway gates (Table 3). Comprehensive account of the factors involved and of the features included in the design of the dam in case of the risk of internal erosion is considered. The possibility of dam failure due to overtopping or military action was eliminated in the report.

Table 2: Hydraulic design data (Swiss Consultants, 1984).

Return period (years)	Peak inflow	Peak outflow (m^3/s)	Peak reservoir WL (m)	Number of spillway gates operable	Fuseplug breached
1000	12000	5650	334.35	3	No
10000	15000	7700	334.5	4	No
PMF	27000	14600	338.45	4	Yes

Table 3: Summary of the discharge capacities for malfunctioning of main spillway (Swiss Consultants, 1984).

Condition	Discharge capacities (m^3/s) at given reservoir water levels		
	338.0 m	**340.0 m**	**340.0 m**
1 gate operable	9626	16281	18156
No gate operable	7525	14030	15870

Several cases were studied in the report concerning the breach in case of dam failure (Table 4). Cases A to D were assuming complete dam failure, while cases E to F consider partial dam failure. It was also assumed that the bottom width of 2 times breach height is expected as the smallest washout, which is imaginable considering enormous flows. The maximum bottom width is expected to be 700 m.

Table 4: Scenarios of breach formation at Mosul Dam (Swiss Consultants ,1984).

Case	A	B	C	D	E	F
Breach width of bottom(m)	700	700	700	700	200	200
Breaching time (hours)	4	4	5	5	2	2
Roughness of the river channel Manning's (n)	0.33	0.050	0.033	0.050	0.033	0.5

The process of breach development according to the model used by the Swiss Consultants includes 4 phases. The first phase includes seepage and internal erosion. This was neglected in the calculations due to the fact that the discharges are very small relative to the power and irrigation releases. Second phase, flow through pipes is formed and get stronger with time. It was simulated by a gate, which gradually opens from (0 to 5) m. Its width equals the weakened zone

(94 m) and its height corresponds to 10% of the seepage zone. The duration of this phase is (0–1) hour. In the third phase, it is assumed that the gate in phase two is simultaneously changing to a breach of the same width and eroded down to the valley bottom. The duration of this phase is (1–3) hours. In the fourth phase, the breach widens to 200 m for cases E and F and to 700 m for cases A to D. The duration is 2 hours for cases E and F, 4 hours for cases A and B and 5 hours for cases C and D. It was also assumed that in the regulating dam and Samarra barrage all the gates are open since 0 hour. Discharges for the reservoir out flow, peak discharges and maximum water level at different locations are given in tables 5 and 6. The peak out flows varies from (551000 and 477000) $m^3.sec^{-1}$. The increase of flow resistance and breaching time causes reduction of peak discharge by 7%, each.

Table 5: Reservoir out flow in 1000 $m^3.sec^{-1}$ (Swiss Consultants, 1984).

Hours/Case	A	B	C	D
0	1	1	1	1
1	13	13	13	13
1.5	80	80	80	80
2.0	215	210	215	212
2.5	372	356	335	325
3.0	474	452	422	404
3.5	535	499	480	453
4.0	551	510	509	475
4.5	538	469	497	460
5.0	507	469	497	460
6.0	405	382	435	405
8.0	271	266	186	278
10.0	186	192	195	198
12.0	123	136	130	142
18.0	37	47	39	49
24.0	18	2	19	22

Table 6: Peak discharges in 1000 $m^3.sec^{-1}$ (Swiss Consultants ,1984).

Case	A	B	C	D
km 00 main dam	551	510	514	477
Km 109.2 Regulating dam	545	503	509	471
Km 117.3 Eski Mosul	481	429	456	409
Km 169.2 Mosul city	405	348	397	342
Km 196.6 Haman Alil	370	308	365	305

Table 7: Maximum water levels (m.(a.s.l.).) (Swiss Consultants ,1984).

Case	A	B	C	D
	m (a. s. l.)			
km 00 main dam	309.5	313.3	307.8	311.7
Km 109.2 Regulating dam	292.8	297.0	292.2	296.6
Km 117.3 Eski Mosul	288.1	291.1	287.6	290.7
Km 169.2 Mosul city	242.8	244.9	242.6	244.7
Km 196.6 Haman Alil	222.7	225.1	222.5	225.0

3. Black and Veatch Review Report and Flood Wave Study

During 2004, Black and Veatch JV. (BV) (Wheeler et al., 2004) were commissioned to carry out a review of The Dam break and Flood wave study for Mosul Dam that was completed in

1984 by the Swiss Consultants (SC) (Swiss Consultants, 1984). The terms of reference were to do the following:

-Assess quality of the data used in the study.
- Check the appropriateness of the software used.
-Comment on the accuracy of the results.
-Identify any significant short coming in the report and the predicted extent of damage that might occur from the dam breach that could affect emergency planning.
-Commenting on any additional information and studies that might be needed to develop an appropriate Emergency Action Plan.

The BV review report; therefore, did not include any "Risk Assessment", but only examined the hazards posed by the presence of the reservoir and outlined what might happen in the event of dam break situation; moreover, the SC report did not have in its scope any damage assessment; although it included a brief discussion of this subject. In Section 2, Mosul Dam and Reservoir of the review report BV summarizes the data given in the SC report regarding the reservoir and catchment area, the design of the dam and construction outline. BV report also describes situations, which did not exist in 1984 and could not have been anticipated by SC at that time. These includes, the dissolution issue in the foundation, which led to the present day on-going program of grouting, the formation of sinkholes, seepage in the left abutment including seepage along both sides of the spillway, and erosion in the bottom outlet plunge pool.
In commenting on the foundation problem, BV states the following:
"It appears from the records of the extended program of grouting that the problems in the foundation are now confined to anhydrite/gypsum layers and relict karst or new karst to be more pronounced in layer gypsum bed (GB0) at depth of 80 – 100 m below the dam foundations". BV commented also on the instrumentation and monitoring of the dam. This was judged by WII/BV team in its site visit in 2004 as follows: "the dam was well instrumented and the observation procedures and reporting were considered to be sound" (Wheeler et al., 2004).

BV proceeded in Section 3 to discuss potential possibilities of failure presented in SC report, which had outlined the main forms of embankment instability that might lead to dam break and agreed with SC statements on this subject, i.e. that such instability may result from:
-External erosion due to overtopping.
-Internal erosion of the embankment or its foundation, due to seepage.
-Instability associated with shear failure within the embankment and foundation or within the embankment alone due to inadequate shear strength.
-Damage from earthquakes or explosions.

In the overtopping scenario, the flood routing calculations of the most severe floods combined with one or more spillway gates being out of order, showed that the dam was safe against over topping judging from the available free board and spillway capacity. Even a dam break would not be possible due to the additional capacity of the fuse plug spillway in the most severe case.

In the consideration of possible failure, due to internal erosion in the embankment or in the foundation; SC put aside such possibility on the grounds that the embankment had a very defensive design by the provision of a wide and very well compacted clay core and ample filters upstream and downstream of the core and provision of drainage zones. And similarly erosion in the foundation would be prevented by the construction of a multiple row grout curtain extending down to a depth of about the maximum height of the dam. BV; however, commented that although some maintenance works on the grout curtain had been expected by SC, the large magnitude of this work was not anticipated or foreseen by SC at the time when they prepared their study.

Finally BV discussed the possibility of dam failure by military action, which was outlined in SC report. In their assumed scenarios, SC had stated that such military action might include the occupation and destruction of the dam by invading troops and decided that they were not qualified to discuss such a strategic situation. On the destruction of the dam by air attack, SC expressed the opinion which was based on discussion with military experts that a significant damage to the dam body requires very large charges and several hits in the same location. The addition of thick rock armor on the dam crest would reduce the damages even more. In addition, the concrete structures were heavily designed and appreciable damage could not be inflicted on. It is worth to mention here that the dam body was hit by an air to ground missile in the 1991 Gulf War with no damage at all, the powerhouse machine hall roof was damaged by another strike and two of the four tall surge tanks were pierced by air missiles. All damages were repaired within six months after the end of the war.

The last scenario examined by SC was that small enemy groups might attack the dam to cause some damages and its ancillary structures. But due to the limited means at the disposal of such groups and their restricted movement coupled with the unlikely event of simultaneous flooding occurring at the same time; such dam break was judge as of very remote possibility. This last assumption proved to be wrong in summer 2014 when ISIS occupied the site and it was only matter of luck that this happened during the dry season and the group did not have time to accumulate enough explosives before they were driven away.

In their assessment of dam failure possibilities, SC concluded that the “largest unknown factor” in Mosul Dam was the underground and the geologic conditions, which were not simple. The conclusion of SC was based on the analysis of the historic frequencies of the types of dam failure and also on local knowledge that a foundation conditions would provide a larger contribution to the overall failure probability than any other single cause. This conclusion was confirmed by BV. As a result, SC judged that, foundation failure as the “least unlikely failure mode” in Mosul Dam. BV considered this to be a reasonable judgment; which was strengthened, if anything, by the scope of the foundation grouting that had been undertaken since the commissioning of the project. BV also concurred with SC’s view that the ultimate size and shape of the breach were not likely to be sensitive to the initial cause of the failure. BV then used different modeling procedures to check SC findings; that was the decoupling of the derivation of the breach hydrograph (which was discussed in chapter 4) from the conditions, which would occur downstream (discussed in Chapter 5).

Chapter 4 was concerned with Modeling of the Reservoir, the Dam and the Breach; BV used DAMBRK UK software, a more modern software, as a check on FLORIS model used by SC for deriving the dam breach hydrograph. The resulting hydrograph had the same value of the peak, but with a broader shape, steeper recession and almost the same volume of the SC hydrograph (Fig.2). It is noticeable; however, that the major part of the flood rise in both cases occurs between 1 and 4 hours. BV then applied, as a check, some empirical formulae derived from case histories of dam failures and which may be used to give an order of magnitude of the floods. The higher estimates by these empirical methods to support the results of both FLORIS and DAMBRK UK models. As a conclusion, BV remarked that the dam breach analysis conducted by SC was sound in 2004 as it had been in 1984.

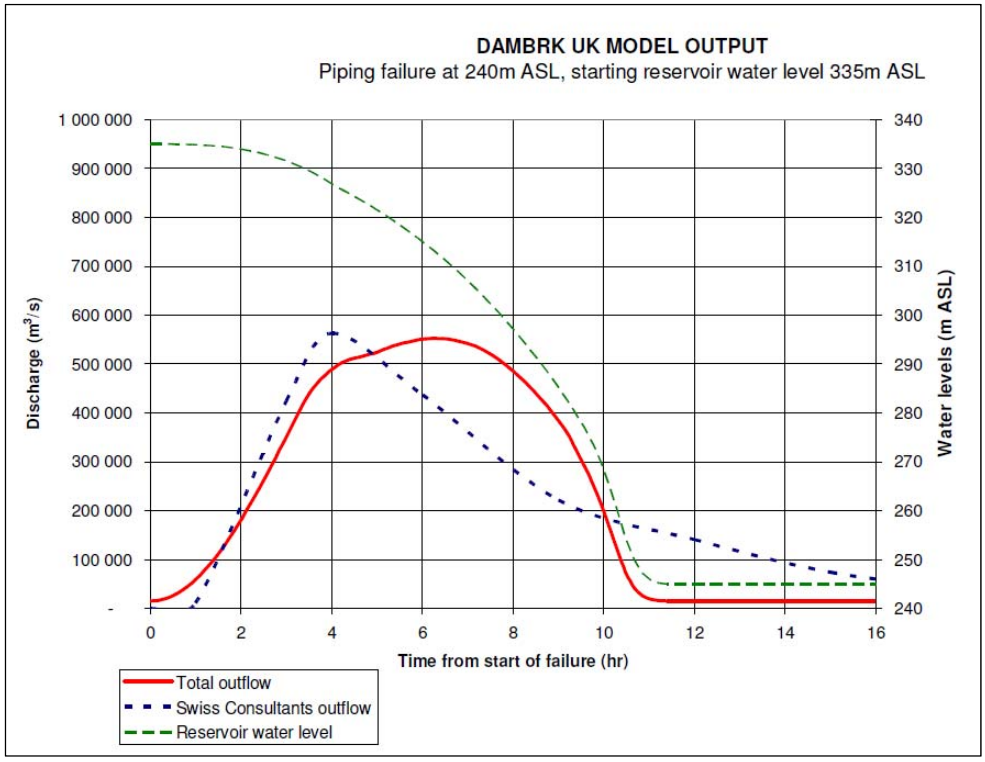

Figure 2: Comparison of dam breach hydrograph by DAMBRK UK and Swiss Consultants (Wheeler, et al., 2004).

Chapter 5 of the BV report was concerned with the modeling of the flood in the downstream valley following the dam break event. BV examined the appropriateness of the use of FLORIS by SC as the modeling software for the Tigris River. But, in the beginning BV explained that at that time they had stopped using DAMBRK UK as a tool for modeling floods in rivers in favor of more modern software due to developments in modeling techniques, especially in the presentation of graphical outputs and interfacing with GIS systems to provide inundation mapping. In this respect, two alternatives were examined by BV, i.e. ISIS version 2.2 (developed by HR Wallingford and Halcrow UK) and MIKEII (developed by the Danish Hydraulic Institute). In comparing with the results of these models with the SC FLORIS results, they showed close coincidence and so BV accepted SC choice as sound and correct.

The next step was checking the schematization of the Tigris River, and examining how the river and its' tributaries were represented and coded and how the structures along its course were introduced, i.e. the Mosul Dam itself including the assumed dam breach, the re-regulating dam, and the Sammara barrage, which controls outflows from the model to the Tharthar Lake, but which breaches when the embankment is overtopped. As a conclusion, BV concludes the following "The schematization of the Mosul Dam model appears to have been carried out in logical manner".

The following stage of checking was on the modelling procedure and how the model was calibrated. BV examined this matter in details and concluded that the choice of the 1964, 1969 and 1974 historical floods for simulation of the model for real floods was acceptable due to the availability of good data of these floods in the form of gauged discharge hydrographs in many stations along the river and its tributaries. And that; since the geometry of the schematized river system was fixed by the cross sections, then the only remaining parameter to be fixed by SC for this calibration was the choice of the Manning "n". The choice by SC of a value of 0.027 gave good results for the mentioned historic floods and the decisions taken by SC to increase this to a higher value of 0.033 for running the dam break wave on the model (as would be expected due to the higher hydraulic resistance expected on the valley sides), and using 0.05 for sensitivity analysis, were good decisions. BV considered that this was realistic and would represent the actual wave flow condition. As a final conclusion, BV judged the whole modelling procedure as acceptable.

Chapter 6 reviewed the consequences of the flood wave. The results derived from the SC-1984 report were checked for consistency; both internally and with the results from other dam break studies. BV found that these results could be accepted with some confidence. This BV review also can be used now as a guide through SC report and to see in summary the key results obtained for Mosul City and Baghdad, which are the major conurbations near the upstream and downstream ends of the model.

Another important point that was highlighted by BV review was the definition of the "Rescue Level" adopted by SC, as the evacuation level which is safe from flooding and therefore, suitable as an evacuation destination. In the upper reaches, where the flow velocities would be high, this was taken as 4 m above the peak of flood wave rounded up to the next whole meter. Reducing it to 2 m in the areas downstream of Baghdad, where the flooded areas are flat and the flow wave depths and velocities are less. BV commented on this choice in subsection 6.4.

In subsections 6.2 and 6.3, BV referred the reader to the parts and annexes in SC report, where the detailed model outputs for the first 100 km downstream were given. BV summed up the main results in a table with the following comment:

What is abundantly clear from the 1984 results for Mosul City that, regardless of the selected assumptions:

- The flood wave may arrive within about 3hours after the initiation of the breach: and;
- The ultimate depth (above normal flood levels in the river) will be 20m.

The warning time, after allowing for any delays in raising the alarm, could therefore, be

extremely short and the effects on the city would be devastating. The whole left bank as far as the ruins of Nineveh would be set under water. Subsection 6.4 also gives the chapters and annexes in SC report for the key results in Baghdad and were summarized in table 5.1 Volume 3 of the 1984's report and reproduced by BV in this subsection with the following comment:

The calculated water levels were reported as 4.6 m higher than the flood level recorded in 1971 flood at a recorded discharge of 4500 $m^3.sec^{-1}$ at the centre of Baghdad (Sari Gauging Station), and estimated as 2.5 m to 3 m higher than the1941's flood, which had been estimated at 7600 $m^3.sec^{-1}$. The results for Baghdad showed that there should be ample time to raise the alarm, achieve an orderly evacuation to avoid heavy losses and mobilize the emergency services. With proper planning there should be opportunity to avoid heavy losses of life. The rescue levels quoted in SC report for Baghdad was between level 38 m (a.s.l.) and 42 m (a.s.l.) with 40 m (a.s.l.) at the centre. BV expressed their concern that due to buildings and other obstructions in the city the rescue levels should be increased by another 2 m to reduce the risks to the evacuation zones.

In chapter 7, On Damage assessment; BV review outlined some of the guidelines used in the UK on the estimation of damage to buildings in the event of flooding; and one method for the estimation of the potential loss of life, which is suggested by the USBR in the United States was also discussed. It is clear from these information if enough warning time is available, then proper emergency planning can also be provided. But in all cases, Mosul City has not enough warning time, which makes a meaningful emergency planning possible.

BV report then goes on to say that damage assessment and emergency planning were not included in SC report. BV in this respect claimed, however, that such a damage assessment and planning were not possible in the short frame of time given to SC in the preparation of the flood wave study and reiterated what SC had already mentioned in their report on describing the damages as *" too large to be estimated at all"*.

In chapter 8, further Modeling; BV investigated the sufficiency and the applicability of the results of the 1984's report. In this context, BV emphasized that SC modeling results were very sound to the degree that no further modeling would be required. But it also could be suggested that some refinements could be introduced in modeling the lower reach around Baghdad in view of the complexity of changes that had occurred in those areas. For such modeling, modern processing and mapping techniques could be utilized. The important conclusion, which BV had reached,; however, is that there was not enough justification for delaying the preparation of the Emergency Action Plan (EAP) even for such refinements. In BV's opinion, the results of SC model are applicable for the preparation of the required EAP, but it was very important that the interpretations of the model results were to be made by an experienced hydraulic engineer, and to produce updated flood maps even if updated surveying work might be required in some places.

Finally in the Conclusion, chapter 9, BV tried to answer three basic questions in order to reach a conclusion, these are;

- Was the dam break model reasonable and realistic representation of the “least unlikely mode of failure”?
- Was the flood wave model using FLORIS rigorous enough in view of the modelling capabilities available in 1984?
- Did SC’s work using FLORIS stand up to examination against software available in 2004?

BV’s answer to these three questions was “YES”.

4. Consequences of Mosul Dam Failure

The seepage problem during the construction and operation periods in Mosul Dam due to the dissolution of gypsum and anhydrite beds raised high concern about the safety of the dam. In view of the situation, the Iraqi Government asked the Swiss Consultants to perform a study about this matter (see Swiss Consultants, 1984). This study was checked again by BV in 2004 (see Wheeler, et al., 2004). Despite the fact that the above studies used different mathematical models, they got the same results. To overcome the problem, grouting operations were the main solution. Later in 2007, the US Corps of Engineers raised high concerns about the safety of the dam and it was reported that in case of the dam failure, it could wipe out whole cities, and was considered to be "the most dangerous dam in the world (see Wakeley et al., 2007; RT, 2014). When ISIS occupied the dam site in 2014, there were fears that they might use explosives to destroy the dam. Numbers of articles were written in this context (RT, 2014; Roulo, 2014; The Independent, 2014; Bender, 2014; Tomkiw, 2014). As an example, RT (2014) wrote “the dam has been suffering a critical lack of maintenance and repair work. And under Islamic State management, a similar situation has been happening to other major dams located on the Euphrates (which still haven’t been freed).

The problem is that when it was built in the mid - 80's, little thought was given to the location. David Petraeus, the former commanding general of the US Army in Iraq, wrote a letter to Prime Minister Nouri Al-Maliki in 2007, indicating the dangers of the soluble soil located underneath the dam, which warrant constant attention and repair work. *“Extraordinary engineering measures” to fill soil gaps and “maintain structural integrity and operating capability of the dam” are a must, according to the US Army Corps of Engineers (USACE) report sent out in the same year. Another report by USACE in 2011 indicated that a failure of the Mosul Dam could cost 500,000 civilian lives in the immediate aftermath”*.

Swiss Consultant (1984) report had traced the wave caused by the dam failure. They calculated the discharges and water levels from the dam site downstream to Baghdad (Table 8 and figure 3). The highest discharges and wave heights are expected to be noticed in the first 122 km downstream the dam. The discharge is expected to be 551000 $m^3.sec^{-1}$ at the start and attenuates to 320 000 $m^3.sec^{-1}$ at the confluence of Tigris- Greater Zab Rivers, which will be reached after 7 hours. The wave height is expected to be 55 m and decreases to 45 m the first 20 km. Mosul city will be affected by the flood after 4 hours of the dam breaching where the maximum water level is expected to be 243 m (a.s.l.) (see Fig. 4). The wave height will be 24 m and it will inundate 74.044 km^2 of the area of Mosul city (Fig. 4). Downstream the confluence with Greater Zab River the discharge of the Tigris River will be reduced to 310 000 $m^3.sec^{-1}$. The

water wave will reach Fatha after 16 hours and the discharge of the Tigris River after its confluence with the Lesser Zab River will be 210000 $m^3.sec^{-1}$. The wave height is expected to be 25 m at Fatha and since the water has to pass through the narrow gap between Hemrin and Makhul Mountains, backwater effect will be noticed and the water velocity at the gap will reach 10 m/s.

Downstream Fatha, the Tigris River valley widens from 1 km to 5 km and the discharge will decrease to 185000 $m^3.sec^{-1}$. At 422 km downstream the dam, the river passes a major city called Tikrit. At that city the discharges will be 185000 $m^3.sec^{-1}$. The wave height will arrive after 22 hours and its height will be 15 m. It will inundate 68.985 km^2 (Table 8, Figures 3 and 4). Then the wave will reach another major city called Samara which is 479 km downstream the dam. The wave time of arrival at this city is 25 hours and height of the wave reaches 10m. The river discharge will be reduced to162000 $m^3.sec^{-1}$. It will inundate 30.100 $km^2$of the city (Table 8, Figures 3 and 4). Further downstream, at a distance of 638 km from the dam, the wave reaches north Baghdad with a height of 4m. The time expected for the wave to reach Bagdad is 38hours. The discharge of the river at this point will be about 46, 000 $m^3.sec^{-1}$. It will take the wave about 10 hours to pass Baghdad and it will inundate an area of about 216.934 km^2 (Table 8, Figures 3 and 4).

Table 8: Discharges, time of arrival and wave height of the wave generated due to Mosul Dam failure (modified after Swiss Consultants, 1984).

Location	Discharge $m^3.sec^{-1}$	Time of Arrival hr.	Wave Height m	Distance km	Flood area Km^2
Dam site	551,000		54	0	
Regulating Dam	545,000	1.3	48	9	
Eski Mosul	481.000	1.6	45	17	
Mosul City	405,000	4	24	69	74.044
Hamam Ali	370,000	5	18	97	
Tikrit	185,000	22	15	422	68.985
Sammara	162,000	25	10	479	30.100
Balad	115,000	28	9	516	
Khalis	81,000	31	6	566	
Tarmiya	72,000	33	4	597	
Baghdad (North)	46,000	38	4	638	
Baghdad (Center)	35,000	44	4	653	216.934
Baghdad (South)	34,000	48	3.5	674	
Diyala Confluence	34,000	>48	3	685	
Salman Pak	31,000	>48	3	708	

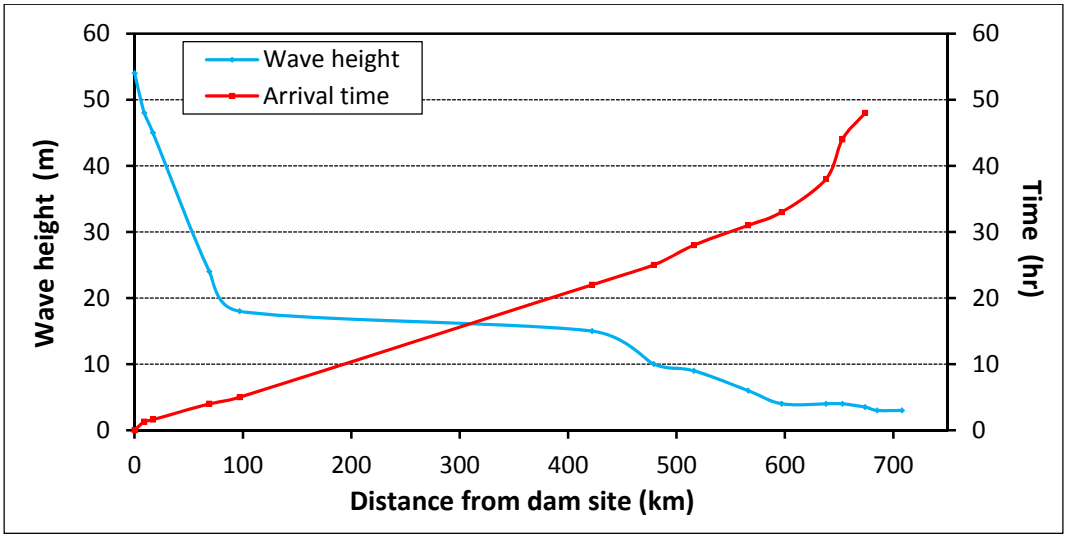

Figure 3: Water wave height due to Mosul Dam failure and time of arrival from the dam site downstream.

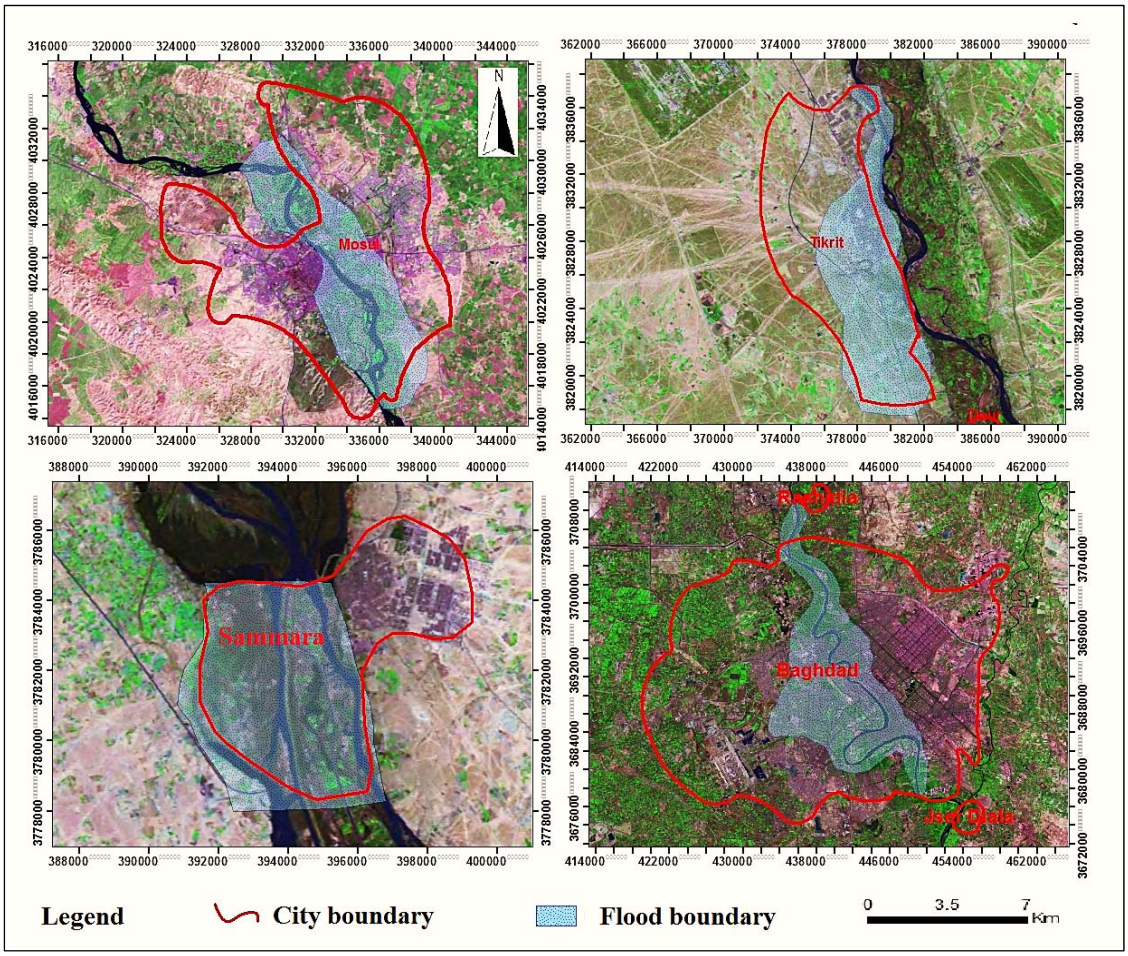

Figure 4: Expected inundation of major cities on the Tigris River due to Mosul Dam failure.

None of the reports gives exact details about the damages expected due to Mosul Dam failure; however, as stated earlier, USACE estimated that the loss of life might reach 500000 persons. In

addition, when we follow the path of the wave from the dam site to Baghdad (685km) we can realize the damage of the infrastructure that the wave is expected to cause keeping in mind that hundreds of towns and villages are located on both banks of the Tigris River.

In view of this situation and to avoid these consequences, the Iraqi Ministry of Irrigation in 1988 decided to build Badush Dam. The site of the dam is located on the Tigris River approximately 40 km downstream from Mosul Dam site and approximately 15 km upstream of Mosul city. The main function of the dam was the protection of the downstream region of the Tigris River valley against the effect of potential Mosul Dam failure, due to any possible reason. In addition, it was also supposed to be used for irrigation and power generation. In the design of the dam, a free volume of 61.5 m height between EL. 245.4 m (a.s.l.) (the normal operation water level), and EL. 307.0 m (a.s.l.) (maximum water level) in case of Mosul Dam failure was allowed. This volume is enough to engulf the flood wave resulting from the worst scenario of Mosul Dam flood wave study. The foundation of Badush Dam is of massive limestone and no dissolution of soluble rock was anticipated. The construction; however, was halted in 1991 due to the economic sanctions imposed on Iraq as a direct result of Iraqis occupation of Kuwait. The completed work is about 40%.

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CHAPTER 9: CONCLUSIONS AND RECOMMENDATIONS

1. Conclusions

Mosul Dam is located on the Tigris River in northwestern Iraq approximately 60 km northwest of Mosul city and 80 Km from Syria and Turkey. It is a multipurpose project for irrigation, flood control and hydropower generation. It is 113 m in height, 3.4 km in length, 10 m wide in its crest and has a storage capacity of 11.11 km^3. The water surface area of the reservoir at the beginning of the dam operation was 380 km^2 with a storage capacity of 11.11km^3 at the maximum operation level 330 m (a.s.l.) including 8.16 km^3 live storage and 2.95 km^3 dead storage. It is an earth fill dam, constructed on bedrocks of Fatha Formation, which consists of gypsum beds alternated with marl and limestone, in cyclic nature. The thickness of the gypsum beds attains 18 m; they are intensely karstified even in foundation rocks. This has created number of problems during construction, impounding and operation of the dam. The topography of the Mosul Dam and reservoir area is characterized by hilly terrain that rise to low mountainous area; representing physiographically Low Mountainous Province. The oldest exposed rocks in the reservoir area belong to the Pila Spi Formation of Late Eocene age while at the dam site; the oldest exposed rocks belong to the Fatha Formation (ex-Lower Fars) of Middle Miocene age. The foundation of the dam lies on this formation. Tectonically, Mosul Dam site and reservoir area are located within the Cham-Chamal Subzone of the Low Folded Zone; within the Unstable Shelf of the Arabian Plate. The project is located far from Zagros Active Seismic Zone, but still many earthquakes were recorded in areas surrounding the project area.

Foundation of Mosul Dam is resting on the rocks of Fatha Formation. The formation is composed of Marls, chalky limestone, limestone, gypsum, and anhydrite, it is highly karstified. The main geologic features under the foundation of the dam are the karstified limestone and the development of solution cavities within the gypsum and anhydrite layers. The right (west) abutment is located in the steeply dipping beds of the Fatha Formation within Butmah East anticline with SE plunge being in the reservoir north of the dam, whereas the left (east) abutment is located on gently dipping beds of the Fatha Formation, which is overlain by fine clastics of the Injana Formation. These differences in lithology as well the dip amount and direction along both abutments as well upstream and downstream of the dam have certainly affected on the hydraulic pressure and increased the dissolution ability of the gypsum and limestone beds, along the abutments and the foundations, which are already karstified in nearby areas.

The first impounding of Mosul Dam reservoir started in the winter of 1986. In September, 1986, seepages and sinkholes development were noticed within the vicinity of the dam. Series of solution channels and sinkholes at the right bank at many points at about 150 meters from the contact with the right abutment of the dam were recognized late 1986. The sinkholes seem to have first manifested themselves in 1992 and settled at a consistent rate until about 1998. The surface expression of the sinkholes suggests that they are caused by underground collapse.

Downstream sinkholes are most likely related to fluctuations in the tail water level of the main dam during operation of the dam and the downstream regulating reservoir. They might be connected to the Darmellah aquifer on the right bank of the reservoir because before the reservoir was impounded, substantial flows on the order of 360 l/sec were encountered from a gypsum breccia layer found during excavation of the tailrace tunnel for the pumped storage scheme to the west of the known sinkholes and the water had a high sulphate content (which was different from the reservoir water). Further evidence for connection of the sinkholes to the Darmellah aquifer is suggested by the appearance of a spring on the right side of the river channel. It appears they are geologically controlled rather than related to seepage. In terms of the impact of sinkholes on the dam, we are not aware of any current technology that could be used to detect the presence or the future development of voids or potential sinkholes beneath the dam embankment structure, either from the surface or from within boreholes.

Six major springs on the left bank at different elevations along the 1.5 km long stretch occurred after the impounding in 1986. It was also noticed that the seepage through the main dam foundations and the left bank increased from 500 l/s to 1400 l/s in that year. It was evident that minerals (mainly gypsum and anhydrite) were leached from the dam foundation at an intensity ranging from 42 to 80 t /day. These minerals were washed out from joints and fractures crossing insoluble rocks and it was expected in view of the repeated maintenance grouting that all gypsum present in joints would be washed away from the zone of grout curtain. Later, the dissolution under the dam and its close proximity also continues at smaller rates due to the nature of the foundation.

During the construction of the dam seepages were noticed under the foundation. To stop this seepage and secure the safety of the dam, an extensive grouting operation took place. The treatment of the Mosul Dam foundations consists of two main types of grouting operations which were blanket grouting and deep grout curtain. The former was performed under the core of the main dam. It was intended to close the opening originally existing in the foundation rocks, creating thereby a more homogenous upper part of the foundation with respect to the permeability and compressibility. The latter was meant to create a barrier against seepage flow in the foundations under the dam and reduce the permeability of the grouted zones to the minimum values possible. The deep curtain was divided into four parts. The first was under the main dam and along its' axis was made by carrying out the usual water pressure tests in the drilled exploratory holes as a first step. The second was under the Saddle dam and fuse plug where two row curtains were carried out. The third place was the left extension curtain which extended over a distance of 1500m. Finally, the forth curtain was performed on the right side extension. This consists of two row curtain in one place (Sec. 123 - 133) and one row curtain in another place (Sec. 134 – 139). Through the period 1986 to 1988 it was noticed that curtains were deteriorating in several places and also lenses of high residual permeability still existed within the Main Dam foundation in the Jeribe limestone and Chalky series. Furthermore, high residual permeability cannot be tolerated in anhydrite/gypsum beds, especially in zones where the dissolution process had started. An extensive maintenance program is necessary to control the seepage process

within the grouted zone to stop dissolution of gypsum and protect the safety of the dam. The total amount of grout injected since 1986 is about 95657 tons.

Despite the grouting operation that took place, acceptable residual permeability was not reached. In addition, new seepage locations were noticed. In view of the situation, studies were conducted to solve the problem. Grout mixes were examined and even methods of delivering and injecting large grout quantities were developed. Sandy mixes were developed by adding certain weight of sand to the cement mix. In addition, pouring gravel after completion of grouting in large takes zones was performed. Continuous inspections showed that using gravel was not effective and very hard to handle. Later, the mix used was Massive grouting where bentonite was added to the mix. The composition of the mixture was as follow: Cement weight = 465 kg, Sand /Cement ratio ≈2:1, So sand weight is 930 kg, Water / Cement ratio =1:1, so water is 415 l, Bentonite -Cement Ratio = 4%, so bentonite weight 18.6 kg. The total dry weight of the mix was 1413.6 $kg.m^{-3}$. For checking of the conditions of the grout curtain and the detection of problematic areas where additional treatment was required piezometric observations were used. Piezometric observation continued and resulted in the repeated application of both normal and massive grouting works up to now as a repair measure where the grout used from 1986 to 2014 reached 95657.43 tons. Massive grouting did not stop the dissolution processes of gysum and anhydrite altogether and it seems that it is not likely to do so in the future. More research work is required to improve massive grout durability. Adding chemicals to the grout, which may interact with gypsum and hinder dissolution? This can help to improve gypsum resistance and increase its stability. Mathematical models might also be used to understand the mechanism of cavities formation and collapsing.

The seepage of water continued and this highlighted the possibility of the dam failure. Different grouting techniques and methods were suggested such as using gelling cement grouts or trying the injection of hot bitumen or diesel oil and bentonite mixture or even the addition of cotton flock or similar fibrous material. Other alternatives involving in the construction of physical barriers were also examined. Ideas suggested involved General filing of seepage paths by precipitation of insoluble materials from ground seepage water or Sealing gypsum/anhydrite surface or using Barriers(this includes blanketing, Positive-Cutoffs, Construction of a new curtain). Other suggestions were strengthening the grout curtain in the problem areas where massive grouting had to be repeated widening the curtain was recommended or to construct a tunnel the length of the chalky series from which grouting would be performed or to construct a series of tunnels and galleries to replace risky material or to construct a diaphragm wall from the upstream berm. The results were the same. Finally, it was decided to limit the maximum operation water level to EL.319m.(a.s.l.) instead of EL.330m.a.s. This recommendation has remained in force up to now with the loss of sizable storage of irrigation water and power potential.

2. Recommendations

Construction work in Mosul Dam started January 25th, 1981 and it started operating on 24th July, 1986. During that period onward, the seepage problem due to the dissolution of gypsum and anhydrite beds under the foundation of the dam could not be stooped. This caused great

concerns about the possibility of dam failure. Several studies were conducted and different models were used to show what is expected to happen in case of the failure of the dam. All the models used showed similar results. It is expected that the wave at the dam site will be 54 m in height and the water discharge is of the order of 551000 $m^3.sec^{-1}$. This wave will reach the capital city of Iraq located 638km downstream the dam. During its course, the wave will inundate several major and small cities. About 74 km^2 of Mosul city will be flooded while at Tikrit and Sammara the area to be inundated reaches about 69 and 30 km^2 respectively. About 216 km^2 of Baghdad will be covered by water. The discharge of the river will be 46000 $m^3.sec^{-1}$ and the wave height will be 4 m. This will cause the death of about halve a million of the civilians as well as great damage to the infra-structure of all the area from the dam site to Baghdad. It is recommended to avoid this catastrophe by finding a solution to the seepage problem. In the same time the construction of the remainder of Badush Dam should be completed according to the original design that allows Badush Dam to hold the wave that could be generated by the failure of Mosul Dam.

In view of the gravity and severity of Mosul Dam situation this requires that very strong and decisive actions should be taken by the government of Iraq, and, in case the government does not have enough resources to be dispensed of on improving the protection of its population, then it should seek the help of the International Community, whether in the form of financial or technical support even if this requires it to be done on diplomatic levels.

The Government of Iraq is urged to take actions on various levels:

A. On the Level of the Site

- To improve the present grouting techniques by supplying new machinery and tools and all infra structures required for better control , compiling and logging of data for a quick and intelligent interpretation, and train the personnel on their use.
- Install many more open pipe piezometers covering the downstream shell of the dam and an extensive area downstream to discover any anomaly in the seepage pattern that might indicate an adverse development and enable an early alert.
- Investigate all available modern techniques and development in ground water movement observation systems and select what fits the situation and acquire them.
- Abandoning the idea of constructing a diaphragm, as all the studies carried out so far proved that such solution is not only not feasible technologically and financially , but it could endanger the integrity of the dam itself. It is understood that the government has signed a letter of understanding with an international company in 2011 to do such work.
- Detailed seismic study of the area should be conducted including micro seismicity to see the effect on old existing faults and possibility of their movement.

B. On the Area Level

- To resume the construction of Badush Dam as it is the only way for the protection of the downstream region in case of Mosul Dam failure. It is understood that the government had revised its plans for this dam to be built as a low run of the river facility and abandon the flood wave protection function.

- To initiate a feasibility study for the decommissioning of Mosul Dam. Such a plan should investigate constructing of a much smaller dam upstream of the present site to provide the projects of North Jazira, South Jazira and East Jazria with irrigation water. The Irrigation water for the middle and southern Iraq irrigation project can be supplied from Al-Tharthar Lake while flood protection of Baghdad can be enhanced by turning Al-Sharee depression north of samara into flood protection facility.

C. On the National Level

- An exact and well defined Emergency Action Plan (EAP) should be prepared by the government. This is a routine action which is taken by all government when building a dam of this magnitude or even less to safe guard the population against such an event, even if the probability of its occurrence is very small. The plan shall be comprised of but not limited to the following:-
  1. Formation of an Emergency action group which could convene at a very short notice and can assume complete authority and responsibility of the situation to take any required action. Needless to say it shall have in its membership representative of all government authorities, civil and military.
  2. Install a modern communication and warning system (independent of any other system) with platforms in all critical points and offices concerned.
  3. Conduct new updated mapping of the downstream region to the last point where the flood wave ceases to have an effect beyond normal flooding. Such mapping should use all modern systems of imagery and GIS systems to define not only inundation limits but to define the safe rescue levels depending on present land use and finally select the required evacuation areas.
  4. The government should promulgate laws and ordinance on the future use of land in the threatened zones. This should also be taken care of in the town planning of the extensions and new developments of the existing cities and townships situated in the river valley.